DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE ANNUAL BULLETIN OF THE BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

(THIS ISSUE INCLUDES ANNOTATED LISTING OF CONTENTS OF ALL ISSUES OF THE BULLETIN FROM VOL. 12 AND TECHNICAL MEMORANDA FROM NO. 106)

VOLUME 16 JULY 1962

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A GRAPHICAL METHOD FOR CHECKING THE DESIGN HEIGHT OF STRUCTURES SUBJECTED TO WAVE RUN-UP

by

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INTRODUCTION

Coastal engineers are often called upon to design protective structures for lake or ocean shores. Some of these structures, especially beach fills and protective dunes, become a part of the nearshore profile of the water basin. In such cases, the existing nearshore profile significantly affects the design of the protective structure, especially the height to which it must be built to prevent overtopping by wind-generated waves. Since the nearshore profiles of most water basins vary considerably in short distances from place to place along the same shore, for most designs so many wave run-up computations are required that it is not feasible to adequately check the design at each change in profile by existing methods. This article presents a method for checking whether or not a structure design is adequate by computing a "critical profile" which, when plotted on transparent paper, can be overlaid on field profile plots to determine if run-up from expected waves will exceed the structure height.

WAVE RUN-UP DATA

Much work has been done to establish the relationships between wave run-up, structure slopes, and wave characteristics. This work has resulted in establishment of some empirical relationships which are shown in Figure 1. In this figure the relationship between the relative wave run-up, R/H_o' (where R is the vertical height to which the wave runs up the slope and H_o' is the deep water wave height corrected for refraction) and the slope of the beach or structure in terms of a function of the deep water wave steepness H_o'/T^2 (T is the wave period) is shown. The relationships shown in this figure have some limitations. First, they are valid only when the waves approach normal to the beach. Second, the beach or structure must be relatively smooth and impermeable. In this respect, the relationships apparently hold for beaches composed of sand which is about 1 mm. or less in median diameter. When the beach material is larger, conservative values of run-up will be predicted by the relationships of Figure 1 and the results may become over-conservative for material sizes larger than 1 mm. Third, the water depth over the toe of the slope must be equal to or greater than three wave heights. The magnitude of the error involved in using the data from Figure 1 for other than expressed limiting conditions will depend upon the extent by which the limitations are exceeded. Other data which may be used to consider relatively large wave heights at the toe of the slope and the roughness of the slope are given in references 1, 2, and 3.

DESIGN CONSIDERATIONS

The factors upon which the design of a protective structure are based may vary considerably. However, usually a "design storm" is selected which fulfills the desired conditions of severity against which the study area is to be protected and the frequency of which is compatible with the expected structure life. Once the design storm and its direction have been determined, the storm surge at the structure site is determined. This surge elevation is added to a selected astronomical tide and any water level increases which may be expected from other sources. The elevation so determined is the "design still water level (SWL)".

In order to determine the height of the proposed structure, the wave run-up for the waves of the "design storm" must be determined. This is usually done by forecasting the "significant wave" of the storm and determining its run-up on the proposed structure using a method such as that given in reference 1. In cases where the structure is being designed for a shore which has varying offshore profiles, an offshore profile typical of the area should be used. When the run-up so determined has been added to the design SWL, a structure height is derived. The structure height derived by this process will not completely prevent overtopping because storm wave heights vary statistically and about 13 percent of the waves will be higher than the significant wave. Thus, run-ups higher than that of the significant wave can be expected (4) and the structure height can be increased to prevent overtopping from as large a percentage of all the waves as is necessary for structure stability or flooding considerations.

When a structure height has been determined in the preceding manner for a typical profile it should be checked for all of the individual profiles in the study area to insure the safety of the entire length of the structure. In addition, the run-up of smaller waves, which break closer inshore, should also be checked because these smaller waves sometimes cause run-up higher than the larger waves breaking in deeper water farther offshore. Using existing methods, checking many wave heights for perhaps a dozen profiles becomes time consuming and, since only spot checks can be made, there is no assurance that the critical wave height, or that height in the height spectrum which produces the largest run-up, has been checked.

SUGGESTED METHOD FOR CHECKING STRUCTURE HEIGHT DESIGN

A proposed dune on a typical profile and an assumed design SWL of +9 feet MLW are shown in Figure 2. In addition, it is assumed that the structure height has been determined to be 3.6 feet higher than the design SWL. This design can be checked for any wave steepness for all profiles by determining a "critical profile" for a maximum run-up of 3.6 feet. This "critical profile" is actually determined by the breaking depths of the waves which produce a run-up of 3.6 feet on various slopes. When this profile has been determined, it can be drawn on transparent paper to the same horizontal and vertical scales to which the study area profiles are

plotted and then superposed on the study area profiles (see Figure 3). When the depths in the study area at any point are greater than the critical depth, the "critical" waves will break closer inshore and the run-up will exceed the structure height. When the depths in the study area at any point are less than those of the "critical profile", the critical waves will break farther offshore and the run-up will not exceed the structure height. Thus, by overlaying the "critical profile" on each of the study area profiles, the structure design height may be checked.

COMPUTING THE CRITICAL PROFILE

A key factor in computing the critical profile is the determination of the wave steepness for use in the run-up calculations. If the storm producing the waves is immediately offshore, the significant wave will have a steepness of approximately 0.20 (see Reference 3, p. 50h) and this steepness may be used in computing the run-up. If the storm producing the waves is farther offshore, the steepness of the significant waves must be determined by using wave forecasting methods to determine the wave decay between the generating fetch and the structure site. When the steepness of the significant wave at the structure site has been determined, it must be converted into its H'_o/T^2 value. Then, the vertical and horizontal scales of the study area profiles must be plotted on transparent paper and the design SWL established as 0 on the vertical scale (see Figure 4). Smooth slopes of 1/40, 1/30, 1/20, 1/10, and 1/5 are laid out through the intersection of the design SWL with the vertical scale at point 0.

The deep water wave height (H'_o) is determined for each slope by entering Figure 1 with the previously determined critical wave steepness and picking off the appropriate value of R/H'_o. Since R is known, 3.6 feet, H'_o can be computed.

Example: $H'_o/T^2 = 0.20$; $R = 3.6$ feet; slope = 1/30

From Figure 1, $R/H'_o = 0.175$ and $H'_o = 3.6/0.175 = 20.6$ feet.

When H'_o and the steepness are known, the breaking depth can be determined from the solitary wave theory as given by Munk(5) and rearranged by Saville(1).

$$d_b = \frac{H'_o}{1.5\left[H'_o/T^2\right]^{1/3}}$$

When d_b has been computed, it is plotted in the proper place on the 1/30 slope (see Figure 4). In the same manner, H'_o and d_b can be determined for the other slopes (extrapolating the curves of Figure 1 for the 1/40 slope) and d_b plotted for each slope. When these critical values of d_b are connected, the "critical profile" results. Critical profiles for a maximum wave run-up of 3.6 feet and wave steepnesses of 0.20 and 0.10 are shown in Figure 4.

USING THE CRITICAL PROFILE OVERLAY

When the critical profile has been plotted, it is superposed on the study area profiles by matching the intersection of the design SWL and the vertical axis on the overlay with the intersection of the design SWL and the structure slope on the study area profile (see Figure 3). If, as in Figure 3, the critical profile for $H_o'/T^2 = 0.20$ always lies below the study area profile, the structure should be safe from overtopping for all waves with $H_o'/T^2 = 0.20$. Also, the structure should be safe from overtopping for all waves of $H_o'/T^2 = 0.10$ even though the critical profile for that wave steepness lies above the study area profile between points (a) and (b) shown on Figure 3. This conclusion is reached because waves which would ordinarily break in the critical area between (a) and (b) will break farther out on the bar crest at (c) and smaller waves will not break until they have passed shoreward of (b) where the study area profile is safe.

In Figure 5, the study area profile is safe for all waves of $H_o'/T^2 = 0.20$ but unsafe for some waves of $H_o'/T^2 = 0.10$. The waves which would overtop the structure in this case would be those that would break between (a) and (b), Figure 5. If the study area profile followed the dotted line shown, the structure would be overtopped by some wave heights of both wave steepnesses shown and the structure would be moved farther shoreward or its height increased in the area of this profile.

In cases where sandy beaches are being considered, it might be wise to consider the possibility of some profile recession during a storm. Some idea of the magnitude of the recession can be obtained from Reference 6.

CONCLUSION

A method has been presented for checking more rapidly the design height of shore structures along a beach fronted by differing offshore profiles. The accuracy of the method is limited by the accuracy of the existing wave run-up relationships and the accuracy of the solitary wave theory as applied to surf conditions. However, when the method is judiciously applied, it offers a quick comprehensive check of the adequacy of structure height to prevent wave overtopping.

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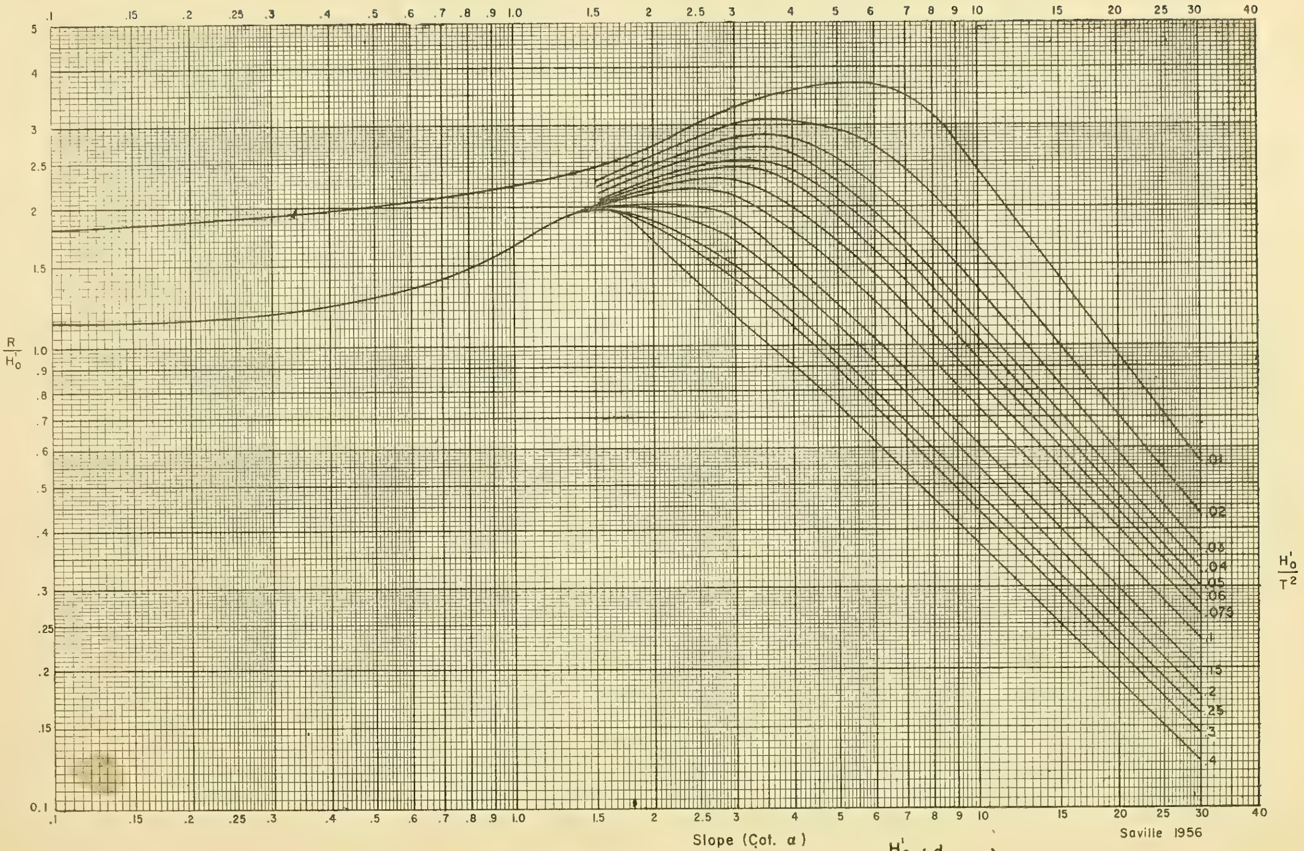


FIGURE 1. WAVE RUN-UP FOR SPECIFIC VALUES OF $\frac{H_o'}{T^2}\left(\frac{d}{H_o'} > 3\right)$.

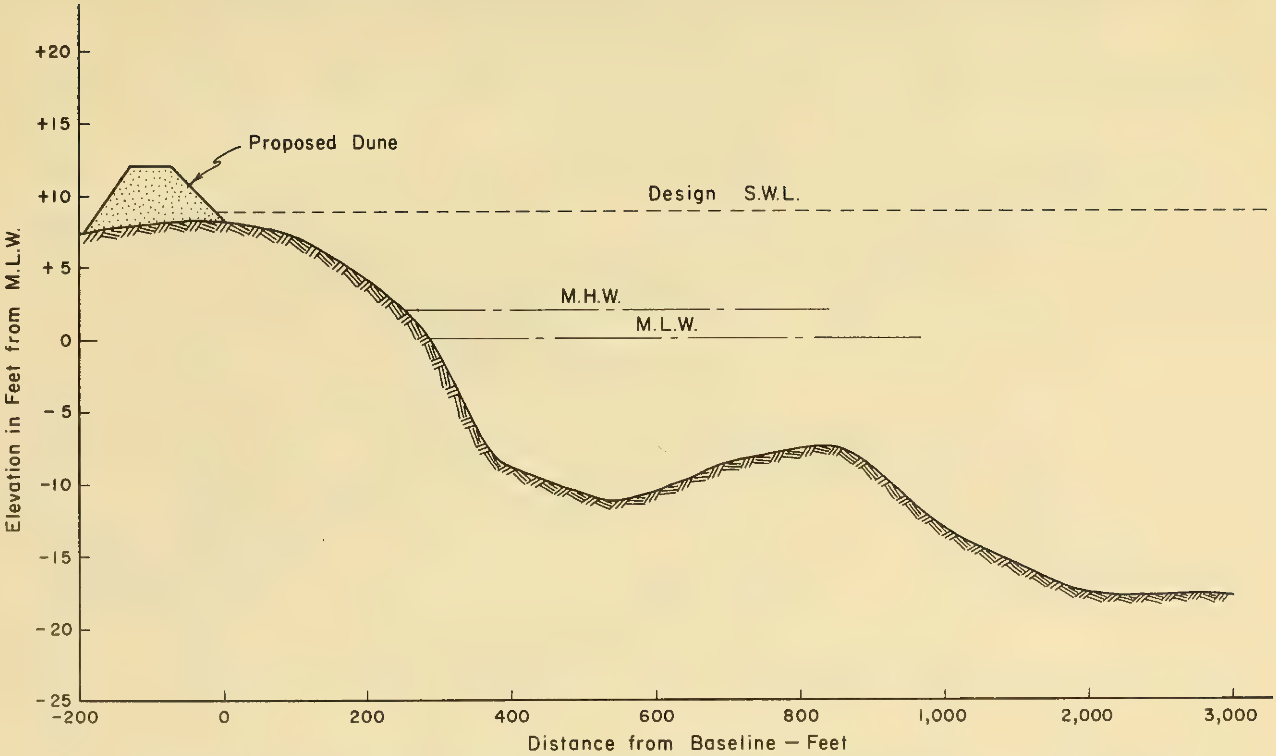


FIGURE 2. TYPICAL NEARSHORE PROFILE SHOWING THE DESIGN STILL WATER LEVEL AND PROPOSED DUNE SECTION

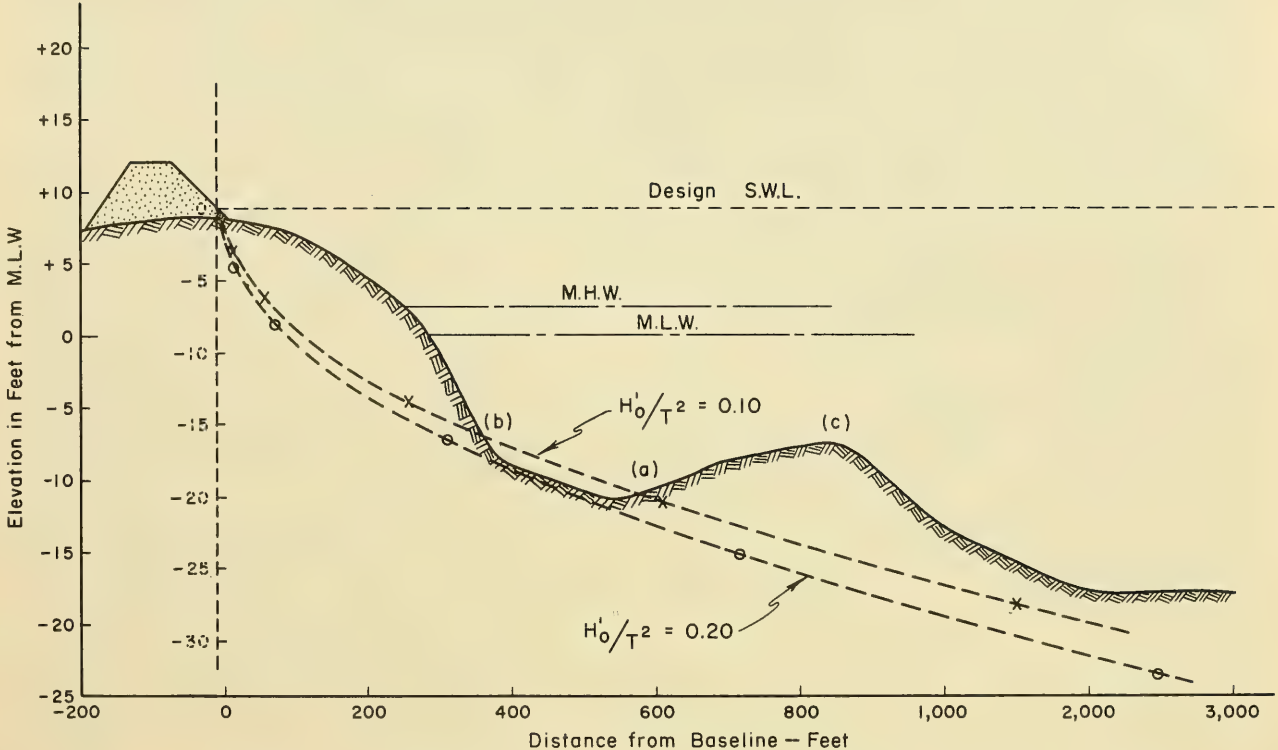


FIGURE 3. CRITICAL PROFILES FOR A MAXIMUM WAVE RUN-UP OF 3.6 FEET OVERLAIN ON A FIELD SURVEY PROFILE

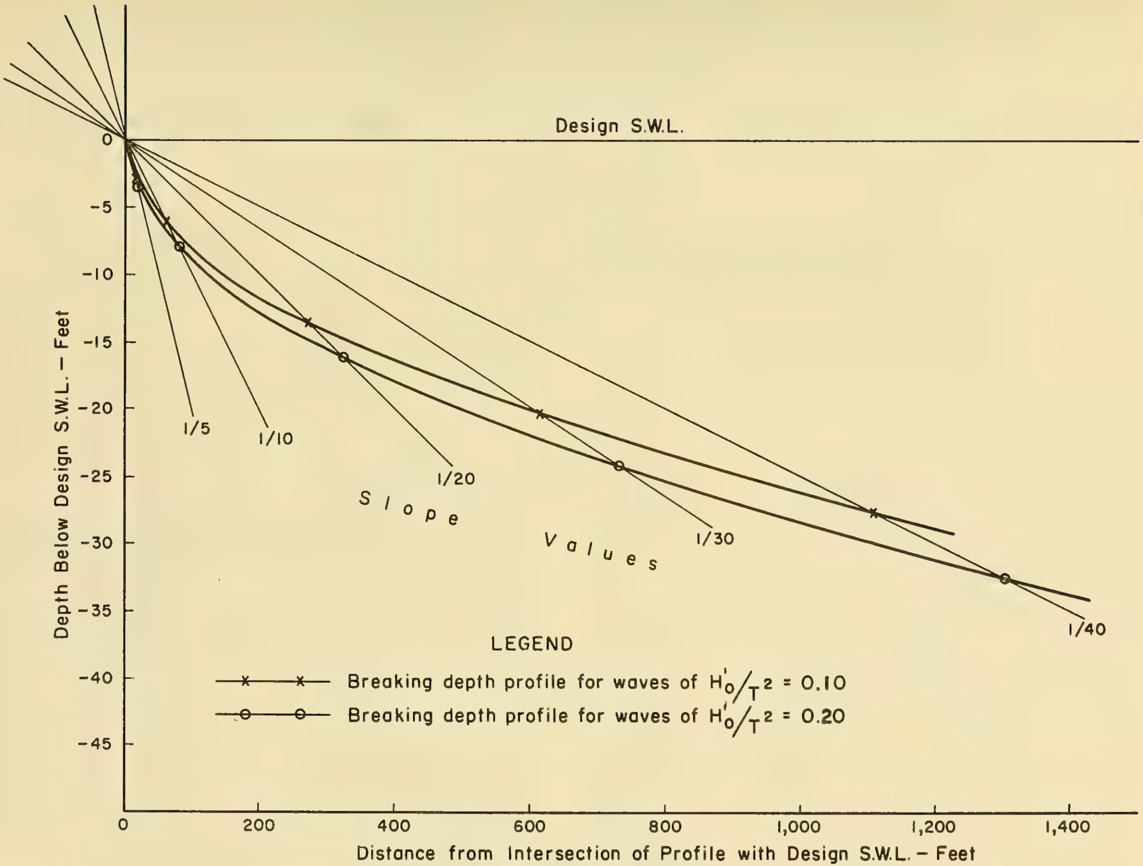


FIGURE 4. CRITICAL PROFILES FOR A MAXIMUM WAVE RUN-UP OF 3.6 FEET

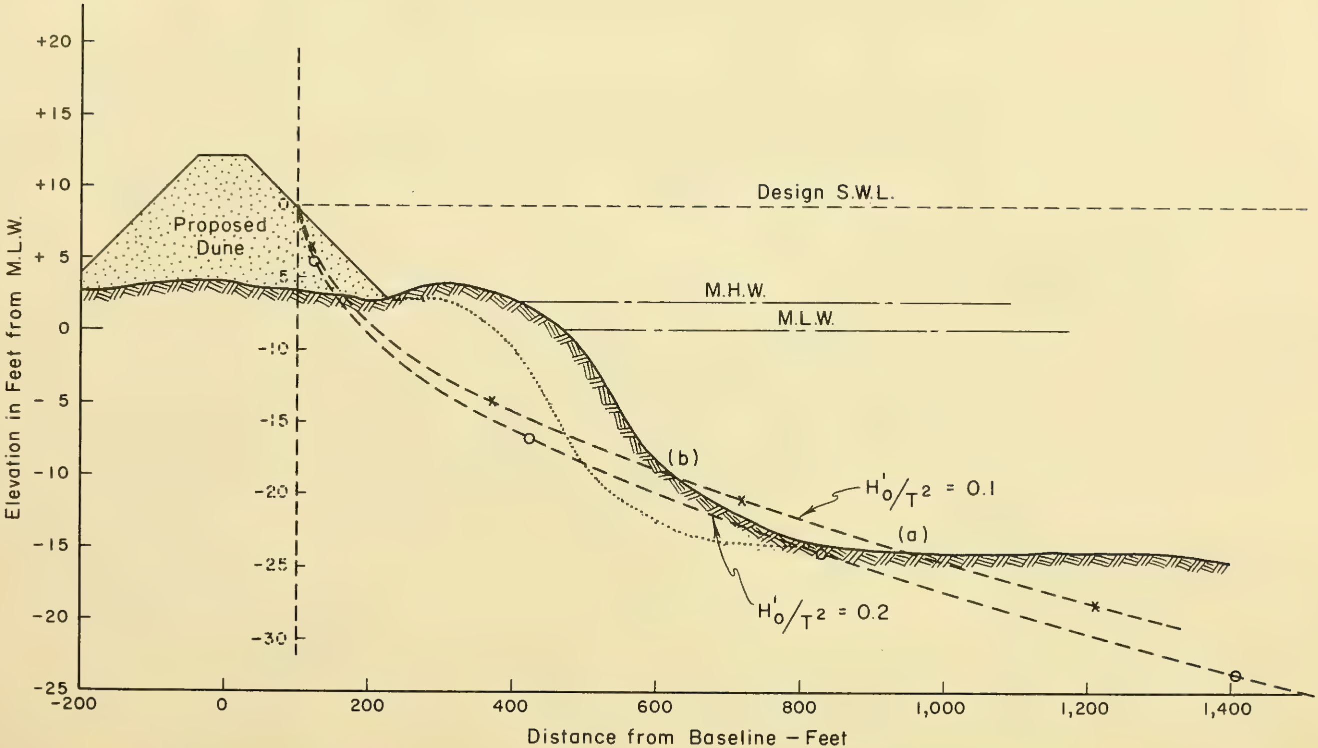


FIGURE 5. CRITICAL PROFILES FOR A MAXIMUM WAVE RUN-UP OF 3.6 FEET OVERLAIN ON A FIELD SURVEY PROFILE

THE NEARSHORE MOVEMENT OF SAND AT DURBAN

by

A. Kinmont

A description of the beach erosion problem at Durban, Natal, South Africa, first appeared in vol. 10 - No. 1 of the Bulletin in July 1956. The following article includes a discussion of the effectiveness of various remedial measures adopted to combat the shore erosion, and first appeared in the published papers (C.S.I.R. Sympos. No. 52, Pretoria, S. A., Mar. 1961) of a Symposium on Marine studies off the Natal Coast arranged by the Natal Regional Research Committee of the South African Council for Scientific and Industrial Research. The paper is reproduced here with the kind permission of the author who is City Engineer at Durban.

INTRODUCTION

The movement of sand off-shore is a problem of vital importance to the City of Durban - from the two aspects of harbour development and beach preservation. Unfortunately these two aspects are antagonistic in that there is no doubt whatsoever that the problem of beach erosion followed upon the development of the Durban Harbour.

The harbour itself is sited within one of the finest natural bays on the continent; however the bay was of a later geological formation than the mainland, for it was formed from the building up, by wind and current action, of the Durban Bluff and the Point, two features which interrupt the general straightness of the Natal coast. The Point spit, representing the youngest formation, is of loose material which possibly began as an off-shore bar, and is held in equilibrium by the different forces and conditions of nature. If this balance is disturbed, then this spit may well tend to disappear and could turn Durban Bay again into an open roadstead. Thus the importance of the stability of the foreshore to the City will be appreciated.

EARLY HISTORY OF HARBOUR

An examination of early Admiralty charts and other documentary evidence shows beyond doubt that the channel depth at the entrance to Durban Bay remained stationary at six feet L.W.O.S.T. from 1685 to 1880, a period of nearly 200 years, while it can safely be assumed that this condition of equilibrium existed for centuries earlier, after the formation of the protecting arms of the Bluff and later the Point.

In 1882, the Innes Breakwater was commenced and with its progress seaward the bar at the Harbour entrance was gradually shifted outwards

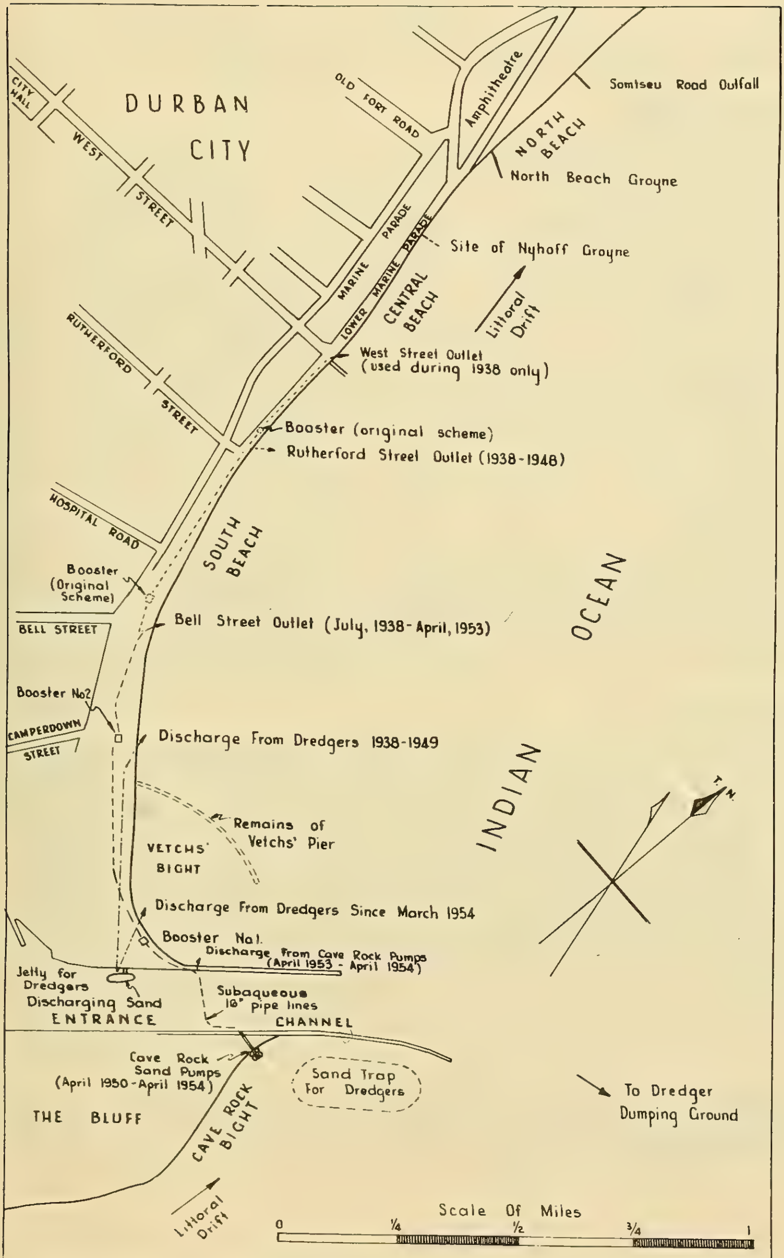
and sand was accumulated at the Cave Rock Bight. Where previously a depth of 40 feet L.W.O.S.T. existed in the Bight, there occurred a shoaling of 11 feet on the completion of the Breakwater. Although there must have followed a period of temporary 'starvation' of the area north of the Breakwater, while this new beach was building up, there was no record of final erosion on the Ocean beaches - on the contrary, the reverse is the case. It appears that the movement of the littoral current was diverted from its normal path merely to take a longer course than formerly before striking the beaches again. In fact, it could be said that the construction of the Breakwater actually had a beneficial effect upon the beaches, since there followed an appreciable accretion of sand on both sides of the piers. During this period, the depth of water at the entrance to the harbour remained in the vicinity of 6 to 8 feet. These facts indicate that there was a constant movement of sand in the general northerly direction.

DREDGING OPERATIONS

With the advent of ocean-going vessels of deeper draught, it became obvious that dredging of the harbour entrance would have to be undertaken to maintain an adequate depth of channel. On account of the steady supply of sand passing the entrance, a bar was always forming there and dredging operations commenced in 1895 with the object of deepening the harbour entrance channel. From that date the entrance level of approximately 18 feet was established and maintained for the ensuing six years, during which period an average of 200,000 cubic yards per annum of sand was removed.

There is no doubt that it is from this period that erosion began to take place on the beaches, increasing in intensity with the dredging operations, as will be shown later in discussing the sand movements on the beaches to the north of the entrance. Gradually the depth of water at the entrance of the harbour was increased and the initial depth of 18 feet in 1895, when a small 500-ton dredger was employed, gradually became 42 feet today, when dredgers of the 3,500-ton class are in operation.

It is of interest to note the various stages of deepening the harbour entrance channel; for example, in the period from 1906 to 1911 when the minimum depth of water available at the bar averaged $31\frac{1}{2}$ feet, the annual removal of sand by dredger amounted to 400,000 cubic yards. This figure was slightly increased during the ensuing decade when the depth was increased to 35 feet, but during the period 1923 to 1931, when the depth at the entrance remained steady at 37 feet, the annual removal of sand averaged 625,000 cubic yards increasing to 650,000 cubic yards in the ensuing six years when the depth was increased by a further foot. From 1939 onwards, the clearance was increased to the present depth of approximately 42 feet; and it is remarkable that over the ensuing 20 years, the quantity of sand which has had to be removed from the Cave Rock Bight sand trap and the harbour entrance has remained at a fairly even average of 800,000 cubic yards per annum.



SAND PUMPING SCHEMES

In passing, it is of interest to note that this figure correlates closely with figures obtained for sand removals from certain other harbours under similar conditions in India and in South America.

STARVATION OF THE BEACHES

The interception of the continuous sand supply from the south by the removal of the sand from the harbour entrance has starved the ocean beaches to the north thereof of their natural source of replenishment, and erosion has consequently taken place. Further - and this is a very serious aspect not generally appreciated - the deepening of the harbour entrance to a depth in the vicinity of forty-five feet has contributed much to the disturbance of the natural "profile of equilibrium" of the whole sand formation. The effect of this will be appreciated when it is realized that, whereas before there existed a feeble slope with a depth approximating eight feet in low water, there is now a very deep and extensive hole exceeding forty-five feet in depth within a distance of the shore that makes the maintenance of the stable sandy slope a matter of great difficulty - if not one of impossibility. This has resulted in a certain amount of deep water erosion, which is borne out by the results of soundings taken over a long period. The early operations of the smaller dredgers could not deal with the total flow passing the harbour entrance, and undoubtedly some of the supply was allowed to pass and continue towards the beaches, but since 1905 - when the larger tonnage vessels were put into commission - an amount which appears to approximate the estimated littoral drift has been removed annually by these vessels and no sand at all has been allowed to pass the harbour entrance. The ocean beaches have therefore been completely starved since that date.

THE VARIATION OF HIGH WATER

The effect of this "sand starvation" is graphically evidenced in the movement of the high water mark over the years under review. Prior to the construction of the entrance breakwaters in 1882, the high water mark had remained reasonably stationary, as far as records are available, but with the construction of the breakwater, an accretion of sand took place north thereof which took the high water mark seawards along a stretch for two miles noth of the breakwater, and an area of some 148 acres was added to the beaches. This is evident from an examination of the plans of 1903, and it is revealing to compare this plan with the survey of 1935 - the period of maximum erosion when there was no attempt at replenishment of wastage by any means. By the latter year, an area of 34½ acres had been lost on the South Beach alone, and over the whole stretch the accretion of earlier years had disappeared.

The movement of the high water line is well illustrated by an examination of Table 1, which indicates the position of H.W.O.S.T. along the line of prolongation of Rutherford Street.

TABLE 1

Year	+ Accretion - Depletion	Comment
1846	0	Earliest records
1903	+ 720 feet	After construction of break-water
1913	- 174 feet	After 10 years regular dredging operations
1935	- 130 feet	Further recession after over 30 years regular dredging
1949	+ 400 feet	Recovery following sand pumping to South Beach

Records of low water over the above periods follow a similar pattern. It is not possible to give similar records for positions along the Central Beach however, as the high water has been restrained by the protection works along that stretch.

WIND DRIFT

In considering the area above L.W.O.S.T., it would be incorrect to ignore entirely the effect of wind drift. This factor is one of some magnitude in Durban, and frequent use is made of graders and bulldozers to recover the sand which has been piled up above H.W.O.S.T., and even further afield. It would seem that wind-blown sand could only be taken from the dry stretches of the beach, but observations indicate that apparently dry sand can be blown in considerable quantities from beach areas still wet from a receding tide - in fact, winds as low as Beaufort Scale 4 (about 15 miles per hour) have moved sand from this moist area. The evaporative effects of the wind are thus important in increasing the erodibility of the sand.

PROFILE OF EQUILIBRIUM

It is not difficult to investigate the movement of sand on the beach itself, but it is more important to know what is happening at the lower levels of the beach profile. Dr. King defined an offshore profile on a well-established sandy beach as a gradually flattening curve seaward

built up by the action of the sea over a long period. When this profile has reached the stage of stability - that is, when the progradation, or the building up of the beach, compensates for any retrogradation, or eroding away - it is usually termed the "profile of equilibrium", and thus it is the ultimate object in any beach reclamation - to restore the profile of equilibrium. It will be appreciated, therefore, that the restoration of such a profile rests largely on the restoration of the lower beach to its correct position and until such a stage is reached, any replenishment of the upper reaches would be of little avail.

UNDERWATER MOVEMENT OF SAND

Much importance, therefore, has been given to the carrying out of regular underwater surveys, either by soundings taken by mechanical devices or by investigations by divers. In this regard, it is important to determine at which depth the maximum movement of sand takes place. In view of the doubts that exist as to whether there is much movement in deep water positions, it is useful to refer to the comprehensive experiments which were carried out by the Beach Erosion Board of the United States to test the effect of offshore dumping in the reclamation of beaches. It was proved conclusively that to be of any material assistance in shore reclamation, dumping of sand must take place within the "breaker" zone, that is, within the sphere of influence of the waves of translation. For example, at the Long Branch, New Jersey experiments (1948 - 1949) sand was deposited in 38 feet of water with negative results. At Atlantic City (1935 - 1942) dumping in 15 to 25 feet depth of water was equally ineffective, while at Santa Barbara, California, (1935-1946) where the conditions generally were very similar to those appertaining to Durban, the experiments failed to establish any accretion of sand when dumped within 20 feet depth of water.

Verification of these facts is borne out by the off-shore dumping of the harbour dredgers at Durban itself. For years the sand taken from the harbour entrance has been dumped at a point about a mile due east of the end of the Innes Breakwater. Today there is a mound on the seabed at a peak depth of eight fathoms and about 40 feet high, representing over 10,000,000 cubic yards of sand. It is obvious that there has been very little movement from this site.

LITTORAL DRIFT

The importance of the "near shore" area in its influence upon beach erosion is therefore obvious, and there is no doubt that the movement of the beach material takes place largely within this zone, motivated by the in-shore currents and the littoral drift, and the effect of these agents has been carefully studied in relation to the Durban problem. The generally accepted theory in regard to this aspect is that the littoral drift, running in a north-easterly direction, is partly induced as an

eddy current by the powerful warm Mozambique Current which flows southwards. It is of interest to note in passing that recent surveys have indicated that this current, generally believed to be in the region of three to four miles offshore, varies considerably in its distance from the coast, and on a recent occasion has been found no nearer than thirty miles off-shore opposite the Bluff Whaling Station. The littoral current referred to travels up the coast in a north-easterly direction and is greatly influenced by the prevailing winds. Assisted by the obliquity of wave action, it carries large quantities of sand in suspension and there is no doubt that, in general, it causes a littoral drift of the sand supply from the south to move up the coast.

EFFECT OF PREVAILING WINDS

The prevailing winds of Durban are north-east and north-north-east on the one hand, and south-west and south-south-west on the other, with periods of calm averaging 21% of the time. These winds have a marked effect upon the wave action and the consequent aggravation of the sand movement. During severe north-easterly winds, for example, the combination of these two natural erosive forces can become so strong as to cause serious depletion of the beach within a very brief period - it is not a rare occurrence for over 100,000 cubic yards of sand to be lost from the beach within a few days during such occasions, particularly when in conjunction with the equinoctial tides. Frequently, the bulk of the 'lost' sand appears to be deposited in off-shore banks parallel to the shore and about 200 to 300 yards out. Some of this material, together with some bed sand, is returned to its original position under the influence of south-westerly winds, due to the fact that the waves on these occasions affect the seabed to a greater degree, and the sand is brought into suspension and moved shoreward by the current, but in general the losses far exceed the later accretion.

EFFORTS TO COMBAT EROSION

Considerable investigation has been directed to the "near shore" area of the beaches, as it is clear that the maximum movement of the sand takes place within this area. It is therefore of interest to examine the positions of the various fathom lines under the different conditions which have existed over the period under review. In order to appreciate the circumstances, however, it will be necessary to give a very brief resume' of the steps which were taken by the Durban Corporation to combat the serious erosion which took place prior to 1938. Several consulting engineers were engaged, and there was unanimity amongst all as to the cause of the erosion, which was agreed to be prolonged dredging in the vicinity of the harbour entrance, and also as to the cure, which was the replenishment of the beaches by artificial means. It was on the method that disagreement arose, and the Council finally adopted the scheme whereby the Railways Administration supplied the sand for replenishment from its dredgers, delivery

being effected through a 42" pipeline to a point on the South Beach opposite Camperdown Road; the first deliveries were made in January, 1938.

Six months later the Corporation installed a special dredger pump by which the sand was picked up at this point and deposited at selected points opposite Bell Street and Rutherford Street. This scheme remained in operation until July, 1949 - a period of eleven years. During this period more than 5,000,000 cubic yards of sand were delivered to Vetch's Bight from the Administration dredgers.

FIRST RESULTS OF SAND PUMPING

An examination of the sand accretion and erosion of the three beaches, namely the South, Central and the North, during the period between 1938 to 1949, is of considerable interest in relation to the effect of the littoral drift. Sand was delivered mainly on the South Beach on the assumption that the northerly drift would take much of it up to the Central and Northern beaches without further pumping. However, it was observed that the bulk of the sand remained on the South Beach, and as the accretion there agreed remarkably closely with the average monthly dredger outputs, it appeared that the sand would not leave this beach in any great quantity. It is therefore reasonable to assume that the anticipated littoral drift northwards did not occur in the vicinity of the South Beach, which confirms a theory advanced by earlier hydrographers to the effect that there is a reverse current in the vicinity of the Vetch's Bight. Although this current is not of any great strength, it is apparently adequate enough to affect the normal flow to the north, so that over the years much of the sand pumped to the South Beach remained in the vicinity.

An examination of the various fathom lines over this period indicates the marked progress made, particularly in the vicinity of the South Beach, where the bulk of the sand was dumped. Booster stations gave additional feed points at Bell Street (1938-49); Rutherford Street (1938-48); and West Street (during 1938 only). Tables 2 and 3 indicate the distance offshore of the various fathom lines, measured from a fixed point.

TABLE 2

At Bell Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
1938	750	1,300	2,450
1949	1,050	2,375	3,750
Gain	+ 300	+1,075	+1,300

TABLE 3

At West Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
1938	450	650	1,500
1949	700	950	2,475
Gain	+ 250	+ 300	+ 975

Similar gains, though of a lesser degree, were experienced opposite the Beach Baths and the Old Fort Road.

THE CAVE ROCK BIGHT SCHEME

Due to the inability of the Railways Administration to maintain an adequate rate of supply from the dredgers to feed the beaches with sand - it had fallen from 70,000 cubic yards per month in 1938 to 27,000 cubic yards per month in 1949 - the Council was forced to consider alternative schemes, and adopted the second of the methods recommended by the consulting engineers. This was the Cave Rock Bight Scheme, whereby a jetty was constructed in the direction of the Cave Rock sand trap, and a pump picked up the sand from the end of the jetty and pumped it to the Camperdown Road area through a fixed line passing under the harbour entrance. This plant, however, had its limitations - the movement of the dredging arm was limited, and it relied upon the natural movement of the sand to replenish the suction zone. The pump, therefore, although its capacity was of the order of 200 cubic yards per hour, was only able to draw sand within narrow limits, and during its four years of operation never exceeded an average of 24,000 cubic yards per month. It will be appreciated that this was an inadequate amount to make up the ravages of the erosive forces in action on the beaches.

EFFECT OF SECOND SCHEME

The results of the Cave Rock Bight Scheme are best illustrated by an examination of the various fathom lines during this period. There were two stages of operation in this scheme, the first from the inception of the scheme early in 1950 to April 1953, during which time the sand supply was boosted to discharge at Bell Street and beyond, and the second stage, when the boosting was discontinued, and the sand was discharged at the base of the North Breakwater - this stage was one of nine months only, but the results were significant, even for such a brief period.

TABLE 4

At Bell Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
1949	1,050	2,375	3,750
April, 1953	1,000	1,650	3,550
Loss	- 50	- 725	- 200
December 1953	1,000	1,350	3,050
Further Loss	-	- 300	- 500

TABLE 5

At Rutherford Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
1949	1,225	1,800	4,150
April, 1953	1,000	1,300	3,400
Loss	- 225	- 500	- 750
December, 1953	1,000	1,150	2,750
Further Loss	-	- 150	- 650

The additional losses incurred between April, 1952 and December, 1953 confirmed that the currents in the vicinity of Vetch's Bight did not carry the sand northwards as anticipated.

THIRD STAGE OF OPERATIONS

With the comparative failure of the Cave Rock Bight Scheme to maintain an adequate rate of sand replenishment to the beaches, the scheme was abandoned in April, 1954, and the dredger supply of the original scheme was reinstated, on the assumption that a supply of the order of that originally

received - namely, about 600,000 cubic yards per annum - would be available to the Council, and that once the beaches had reached some form of stability, this supply would continue at a rate in the vicinity of 300,000 cubic yards per annum. During the first year of the third stage, nearly 400,000 cubic yards of sand had been supplied, and the losses of the second stage had been converted to small gains. Unfortunately, for various reasons, the Railways Administration was unable to maintain the expected supply of sand, and the supplies gradually diminished until barely 50,000 cubic yards have been pumped in the past twelve months, and as must be expected, recession of the beaches is again taking place. This can be seen in the following figures:

TABLE 6

At Bell Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
December, 1953	1,000	1,350	3,050
End 1955	1,010	1,925	3,600
Gain	+ 10	+ 575	+ 550
End 1959	995	1,505	3,300
Loss	- 15	- 420	- 300

TABLE 7

At Rutherford Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
December, 1953	1,000	1,150	2,750
End 1955	990	1,395	3,050
Loss/Gain	- 10	+ 245	+ 300
End 1959	990	1,345	2,930
Loss	-	- 50	- 120

TABLE 8

At West Street

Year	Distance of fathom lines in feet		
	Two	Three	Four
December, 1953	650	850	1,100
End 1955	525	855	1,100
Loss/Gain	- 125	+ 5	-
End 1959	515	825	1,060
Loss	- 10	- 30	- 40

In considering these tables, it should be pointed out that the sections opposite Bell Street and Rutherford Street refer to a comparatively flat profile of the seabed, and that the profile steepens sharply as the sections move north. Thus, it requires only a small difference in sand level on the South Beach to move a contour considerably one way or another, whereas this does not apply to the Central and North Beaches.

Further north the beaches have been steadily losing ground, even in the vicinity of the groynes which have been erected on the northern beaches.

POSITION OF GROYNES

In regard to these structures, several interesting facts have developed in connection with the movement of the sand. These groynes - built more in the form of breakwaters, rather than in the usually accepted form of groyne - were constructed during the period 1954 to 1956, and therefore have been in existence over five years. When constructed, the depth of water at the end of the breakwaters averaged about ten to twelve feet, but over their period of existence, sand has been eroded from the foot of the structures to such a degree that at places a depth of water of over 30 feet has been found. There is no doubt that eddy currents are being established in this vicinity, probably due to the impermeability of the structure, and the inability of the waves to pass over, and a new aspect of the erosion problem has been set up. Due to this depth of water, the normal gradients of the beach have, in turn, been disturbed, and the sand is taking up an unusual pattern for inter-groyne build-up. Instead of accumulating on the southern face of the groyne - the littoral drift being northerly - the sand is forming a central spit, and the areas on both sides of the groynes are being

severely eroded, with consequent threat of movement of the structure itself, and, incidentally, the formation of dangerous bathing conditions.

FUTURE DEVELOPMENTS

From the experiences of the past twenty-four years, during which the City Council has endeavoured to preserve its ocean beaches against erosion, it is clear that artificial means of beach replenishment must be continued, and that the minimum quantities to be provided by these means must be of the order of 300,000 cubic yards per annum. In an effort to recover lost ground, this quantity should be considerably augmented in the near future, up to 600,000 cubic yards per annum.

Together with this artificial supply, adequate and efficient groynes must be built to hold the sand and, once the whole position of equilibrium has been restored, it should be possible to reduce the artificial feed to a nominal quantity.

The most important aspect which requires further research is the question of the sand-retaining structures. The form, dimensions and placing of the future groynes can only be determined after considerable research, and the final conclusion may be quite a radical departure from the normally accepted form - particularly in regard to direction.

SURF CLIMATE AT THREE SELECTED U. S. COASTAL LOCALES - ATLANTIC CITY, NEW JERSEY; HILLSBORO INLET, FLORIDA; YAQUINA BAY, OREGON

by

Johnny A. Hall
Beach Erosion Board

INTRODUCTION

A program of visual surf observation to provide fundamental information on surf characteristics along the coasts of the United States was initiated on a cooperative basis between the Beach Erosion Board and the United States Coast Guard in April 1954. This project has been designated as the Cooperative Wave Observation Program (CWOP). Originally observations were made at 27 stations, but the number of stations has now been reduced to 18 for various reasons. The locations of the stations, both the active and the inactive, are shown on Figure 1.

OBSERVATION AND COLLECTION OF DATA

Each Coast Guard Station participating in the CWOP makes visual observations every 4 hours unless adequate observation is prevented by poor visibility. The information is recorded on a standarized form and consists of the following: a) breaker period in seconds, b) significant breaker height to nearest one-half foot, c) direction from which wave approaches before breaking, d) the type (spilling, plunging or surging) of breakers formed, and e) any unusual meteorological or oceanological conditions such as strong winds, abnormal tides, etc. The tabulated data are mailed weekly to the Beach Erosion Board. While realizing that data such as this should be ideally collected with a precision beyond the capabilities of human observers, it is felt by those associated with the program that errors in the collected data are not excessive and are statistically compensated over a few months of observations.

AUTOMATIC DATA PROCESSING

The difficulty of manually transforming the great mass of data thus collected into useful statistical summaries became apparent during the preparation of Beach Erosion Board Technical Memorandum No. 108, Surf Statistics for the Coasts of the United States, dated November 1958. Compilation of the statistical tables for that memorandum from CWOP data took approximately one man-year. To efficiently expedite the reduction of the CWOP information, consideration was given to Automatic Data Processing (ADP) by machine. Data was selected from three stations - Atlantic City Lifeboat Station, Atlantic City, New Jersey; Hillsboro Inlet Light Station, Pompano Beach, Florida; and Yaquina Bay Lifeboat Station, Newport, Oregon -

FIGURE I. U.S. COAST GUARD STATIONS PARTICIPATING IN COOPERATIVE WAVE OBSERVATION PROGRAM

as the first to be automatically processed (on a trial basis). These particular stations were chosen because the two stations on the Atlantic Coast are located near where cooperative beach erosion control studies have been made or are in process, and near existing Beach Erosion Board wave gauges, wave data from which could be correlated with the data taken by visual observations. At the time of this writing, correlation in this manner had not yet been accomplished. The station on the west coast was selected because of its exposure to a wide range of surf heights and periods. Data from these three stations for the 5-year period 1 January 1955 through 31 December 1959, were given to a contractor for automatic processing on computing machines. The specifications of the contract required that the observations for each day be punched and verified on a single 80-column "Hollerith" card, that a program for processing be written and that the Beach Erosion Board be furnished a hard copy of the computer program and the following processed data: a) distribution of occurrence of breakers by height and direction for each calendar month and for all observations; b) distribution of occurrence of breakers by period and direction for each calendar month and for all observations; c) distribution of occurrence of breakers by height and period for each calendar month and for all observations; d) cumulative frequencies for each month and for all observations expressed as percentages; and e) distribution of breaker height by type for all observations. A Univac SS-80 computer was utilized by the contractor to make the statistical computations.

PRESENTATION OF SURF STATISTICS

Tables 1 through 9 present surf statistics compiled by ADP at the three selected stations. These tables do not represent the entire statistical output from the automatic processing, but only selected statistics for the over-all (5-year) period. It is not intended in this paper to offer an analysis of the data, but certain facts are readily apparent. Yaquina Bay (Tables 7 through 9) is exposed to surf with greater mean heights and periods and has a greater range of both heights and periods than either Atlantic City (Tables 1 through 3) or Hillsboro Inlet (Tables 4 through 6). The direction of wave approach at Yaquina is also indicated to be more constant with over 93 percent of observed occurrences arriving from the west. Likewise it is fairly obvious from Tables 1 through 6 that the frequency distributions of breakers by height alone at Atlantic City and Hillsboro Inlet are very similar, but breaker periods are longer on the average at Hillsboro Inlet. Also deep water waves were observed most frequently approaching Hillsboro Inlet from the southeast while at Atlantic City the most frequent direction of approach was from the east.

The tabulated data for surf statistics at the three selected locations are presented herein as possibly being of use and interest to many of the readers of the BULLETIN. The cooperative program of surf observations is continuing, and it is hoped that similar or even more extensive data may be published in the future for these and other locations on the coasts of the United States.

TABLE 1
ATLANTIC CITY, N. J.

Distribution of Breaker Heights by Periods - 1 January 1955 through 31 December 1959

Ht. (ft.) \ Per. (sec.)	(1)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	TOTALS
0-1	#	19	47	207	1219	612	73	7	1	1	2186
	%	0.41	1.00	4.42	26.04	13.07	1.56	0.15	0.02	0.02	46.69
1-2	#	18	45	123	832	504	54	3	1		1580
	%	0.38	0.96	2.63	17.77	10.77	1.15	0.07	0.02		33.75
2-3	#	17	22	46	309	173	33	1			601
	%	0.36	0.47	0.98	6.60	3.70	0.71	0.02			12.84
3-4	#	1	6	29	111	36	14				197
	%	0.02	0.13	0.62	2.37	0.77	0.30				4.21
4-5	#	1	1	9	22	27	8	1			69
	%	0.02	0.02	0.19	0.47	0.58	0.17	0.02			1.47
5-6	#			4	8	9	3				24
	%			0.09	0.17	0.19	0.06				0.51
6-7	#				3						3
	%				0.06						0.06
7-8	#				2	2					4
	%				0.04	0.04					0.09
8-9	#										
	%										
9-10	#						1				1
	%						0.02				0.02
TOTALS	#	56	121	418	2506	1363	186	12	2	1	4682
	%	1.20	2.58	8.93	53.52	29.11	3.97	0.26	0.04	0.02	

(1) # = Number of observations
% = Percent of total observations

TABLE 2

ATLANTIC CITY, N. J.

Distribution of Breaker Heights by Direction - 1 January 1955 through 31 December 1959

Dir. \ Ht. (ft.)	(1)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	TOTALS
NE	#	269	255	166	59	20	10	2	3			784
	%	5.76	5.46	3.56	1.26	0.43	0.21	0.04	0.06			16.80
E	#	201	129	42	17	6	2	1				398
	%	4.31	2.76	0.90	0.36	0.13	0.04	0.02				8.53
SE	#	1380	981	311	103	41	10		1		1	2828
	%	29.57	21.02	6.66	2.21	0.88	0.21		0.02		0.02	60.60
S	#	97	91	31	12	1	1					233
	%	2.08	1.95	0.66	0.26	0.02	0.02					4.99
SW	#	234	120	53	7	2	2					418
	%	5.01	2.57	1.14	0.15	0.04	0.04					8.96
TOTALS	#	2181	1576	604	198	70	25	3	4		1	4661
	%	46.73	33.77	12.94	4.24	1.50	0.54	0.06	0.09		0.02	

(1) # = Number of observations

% = Percent of total observations

TABLE 3

ATLANTIC CITY, N. J.

Distribution of Breaker Periods by Direction - 1 January 1955 through 31 December 1959

Dir. \ Per. (sec)	(1)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	TOTALS
NE	#	6	15	42	443	247	30				783
	%	0.13	0.32	0.90	9.53	5.31	0.65				16.84
E	#	6	17	34	211	117	8	2			395
	%	0.13	0.37	0.73	4.54	2.52	0.17	0.04			8.50
SE	#	31	76	298	1538	763	107	7	1	1	2822
	%	0.67	1.64	6.41	33.08	16.41	2.30	0.15	0.02	0.02	60.70
S	#	4	3	18	109	90	7	1			232
	%	0.09	0.07	0.39	2.35	1.94	0.15	0.02			4.99
SW	#	9	10	26	196	137	31	1	1		411
	%	0.19	0.22	0.56	4.22	2.95	0.67	0.02	0.02		8.84
TOTALS	#	56	121	418	2497	1354	183	11	2	1	4643
	%	1.21	2.60	8.99	53.71	29.12	3.94	0.24	0.04	0.02	

(1) # = Number of observations
% = Percent of total observations

TABLE 4
HILLSBORO INLET, FLORIDA

Distribution of Breaker Heights by Periods - 1 January 1955 through 31 December 1959

Ht. (ft.) \ Per. (sec.)	(1)	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	TOTALS
0-1	#	12	256	863	1847	1974	361	45	10	3	3	2	5376
	%	0.12	2.45	8.26	17.68	18.90	3.46	0.43	0.10	0.03	0.03	0.02	51.47
1-2	#	3	75	527	1304	790	106	27	6	5	2		2845
	%	0.03	0.72	5.05	12.48	7.56	1.02	0.26	0.06	0.05	0.02		27.24
2-3	#		28	351	622	353	63	21	7	2			1447
	%		0.27	3.36	5.95	3.38	0.60	0.20	0.07	0.02			13.85
3-4	#		19	164	211	117	27	10	8	6			562
	%		0.18	1.57	2.02	1.12	0.26	0.10	0.08	0.06			5.38
4-5	#		1	63	52	37	10	2	2	1	1		169
			0.01	0.60	0.50	0.35	0.10	0.02	0.02	0.01	0.01		1.62
5-6	#			10	15	9	1						35
	%			0.10	0.14	0.09	0.01						0.34
6-7	#		1		4	1							6
	%		0.01		0.04	0.01							0.06
7-8	#			1	1	4							6
	%			0.01	0.01	0.04							0.06
TOTALS	#	15	380	1979	4056	3285	568	105	33	17	6	2	10446
	%	0.14	3.64	18.95	38.83	31.45	5.44	1.01	0.32	0.16	0.06	0.02	

Note: 27 observations recorded calm seas and are not included in the Table.

(1) # = Number of observations
% = Percent of total observations

TABLE 5
HILLSBORO INLET, FLORIDA

Distribution of Breaker Heights by Direction - 1 January 1955 through 31 December 1959

Dir.	Ht. (ft.) (1)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	TOTALS
NE	#	417	405	244	151	66	24	3	1	1311
	%	3.99	3.88	2.34	1.45	0.63	0.23	0.03	0.01	12.55
E	#	3760	1803	929	341	84	9	1	5	6932
	%	36.00	17.26	8.89	3.26	0.80	0.09	0.01	0.05	66.36
SE	#	1191	628	271	70	19	2	2		2183
	%	11.40	6.01	2.59	0.67	0.18	0.02	0.02		20.90
S	#	8	9	3						20
	%	0.08	0.09	0.03						0.19
TOTALS	#	5376	2845	1447	562	169	35	6	6	10446
	%	51.47	27.24	13.85	5.38	1.62	0.34	0.06	0.06	

Note: 27 observations recorded calm seas and are not included in the Table

(1) # = Number of observations
% = Percent of total observations

TABLE 6

HILLSBORO INLET, FLORIDA

Distribution of Breaker Periods by Direction - 1 January 1955 through 31 December 1959

Dir. \ Per. (sec.)	(1)	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	TOTALS
N	#	1	22	211	478	441	98	31	16	1	1	1	1311
	%	0.01	0.21	2.12	4.58	4.22	0.94	0.30	0.15	0.01	0.01	0.01	12.55
E	#	5	214	1210	2654	2326	415	70	16	16	5	1	6932
	%	0.05	2.05	11.58	25.41	22.27	3.97	0.67	0.15	0.15	0.05	0.01	66.36
SE	#	9	143	545	918	509	54	4	1				2183
	%	0.09	1.37	5.22	8.79	4.87	0.52	0.04	0.01				20.90
S	#		1	3	6	9	1						20
	%		0.01	0.03	0.06	0.09	0.01						0.19
TOTALS	#	15	380	1979	4056	3285	568	105	33	17	6	2	10446
	%	0.14	3.64	18.95	38.83	31.45	5.44	1.01	0.32	0.16	0.06	0.02	

Note: 27 Observations recorded calm seas and are not included in the Table.

(1) # = Number of observations
% = Percent of total observations

TABLE 7
YAQUINA BAY, OREGON

Distribution of Breaker Heights by Periods - 1 January 1955 through 31 December 1959

Ht. (ft.) \ Per. (sec.)	(1)	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	TOTALS
2-3	#							4	25	54	49	17	71	148	9	1	378
	%							0.07	0.41	0.89	0.81	0.28	1.17	2.45	0.15	0.02	6.25
3-4	#			1	2	2	5	6	131	279	98	58	451	772	2		1807
	%			0.02	0.03	0.03	0.08	0.10	2.17	4.61	1.62	0.96	7.45	12.76	0.03		29.86
4-5	#			9	4	10	27	32	111	159	144	225	447	398	4		1570
	%			0.15	0.07	0.17	0.45	0.53	1.83	2.63	2.38	3.72	7.39	6.58	0.07		25.95
5-6	#	1	1	17	7	26	93	60	74	123	251	211	221	112	1		1198
	%	0.02	0.02	0.28	0.12	0.43	1.54	0.99	1.22	2.03	4.15	3.49	3.65	1.85	0.02		19.80
6-7	#				5	8	19	8	52	100	182	89	70	30			563
	%				0.08	0.13	0.31	0.13	0.86	1.65	3.01	1.47	1.16	0.50			9.30
7-8	#					2	3	4	21	29	65	26	18	10	1		179
	%					0.03	0.05	0.07	0.35	0.48	1.07	0.43	0.30	0.17	0.02		2.96
8-9	#			1			1	1	11	20	64	20	10	7			135
	%			0.02			0.02	0.02	0.18	0.33	1.06	0.33	0.17	0.12			2.23
9-10	#								3	1	14	6	2	3			29
	%								0.05	0.02	0.23	0.10	0.03	0.05			0.48
10-11	#							2	4	15	40	6	4	1			72
	%							0.03	0.07	0.25	0.66	0.10	0.07	0.02			1.19
11-12	#									4							4
	%									0.07							0.07
12-13	#								1	5	8	3					17
	%								0.02	0.08	0.13	0.05					0.28
13-14	#										3						3
	%										0.05						0.05
14-15	#									2	3						5
	%									0.03	0.05						0.08
15-16	#					1	3	2	2	11	22	2					43
	%					0.02	0.05	0.03	0.03	0.18	0.36	0.03					0.71
16-17	#							1									1
	%							0.02									0.02
TOTALS	#	1	1	28	18	49	151	120	435	798	947	663	1294	1481	17	1	6051
	%	0.02	0.02	0.46	0.30	0.81	2.50	1.98	7.19	13.19	15.65	10.96	21.39	24.48	0.28	0.02	

Note: 32 Observations recorded SURF too high to fit the parameters of the machine calculation and are not included in the Table.
15 Observations recorded SURF too low to fit the parameters of the machine calculation and are not included in the Table.
15 Observations recorded CALM SEAS and are not included in the Table.

(1) # = Number of observations
% = Percent of total observations

TABLE 8

YAQUINA BAY, OREGON

Distribution of Breaker Heights by Direction - 1 January 1955 through 31 December 1959

Dir.	Ht. (ft.) (1)	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	TOTALS
S	#				1	1											2
	%				0.01	0.01											0.03
SW	#	1	1	12	23	42	10	4		1		2			2	1	99
	%	0.01	0.01	0.16	0.30	0.54	0.13	0.05		0.01		0.03			0.03	0.01	1.28
W	#	388	1932	1728	1329	763	360	329	65	175	4	66	7	9	107		7262
	%	5.02	25.01	22.37	17.20	9.88	4.66	4.26	0.84	2.27	0.05	0.85	0.09	0.12	1.39		94.01
NW	#		9	81	179	38	3	2		1					2		315
	%		0.12	1.05	2.32	0.49	0.04	0.03		0.01					0.03		4.08
TOTALS	#	389	1942	1821	1532	844	373	335	65	177	4	68	7	9	111	1	7725
	%	5.63	25.14	23.57	19.83	10.93	4.83	4.34	0.84	2.29	0.05	0.88	0.09	0.12	1.44	0.01	

Note: 32 Observations recorded Surf too high to fit the parameters of the machine calculations and are not included in the Table.
15 Observations recorded Surf too low to fit the parameters of the machine calculation and are not included in the Table.
15 Observations recorded Calm Seas and are not included in the Table.

(1) # = Number of observations
% = Percent of total observations

TABLE 9
YAQUINA BAY, OREGON

Distribution of Breaker Periods by Direction - 1 January 1955 through 31 December 1959

Dir. \ Per. (sec.)	(1)	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	TOTALS
SW	#			1		2	4	6	7	17	16	18	6				77
	%			0.02		0.03	0.07	0.10	0.12	0.28	0.27	0.30	0.10				1.28
W	#	1	1	10	8	16	73	84	399	751	902	612	1280	1479	17	1	5634
	%	0.02	0.02	0.17	0.13	0.27	1.22	1.40	6.65	12.51	15.03	10.20	21.33	24.64	0.28	0.02	93.87
NW	#			16	10	31	74	30	29	30	29	32	8	2			291
	%			0.27	0.17	0.52	1.23	0.50	0.48	0.50	0.48	0.53	0.13	0.03			4.85
TOTALS	#	1	1	27	18	49	151	120	435	798	947	662	1294	1481	17	1	6002
	%	0.02	0.02	0.45	0.30	0.82	2.52	2.00	7.25	13.30	15.78	11.03	21.56	24.68	0.28	0.02	

Note: 15 Observations recorded Calm Seas and are not included in the Table.

(1) # = Number of observations
% = Percent of total observations

LIST OF CONTENTS FOR
PROCEEDINGS OF COASTAL ENGINEERING CONFERENCES

For the convenience of readers of the ANNUAL BULLETIN, many of whom presumably have occasion to refer to the various papers published in proceedings of the several conferences on coastal engineering sponsored by the Council on Wave Research, a listing has been compiled and furnished by Dr. Thorndike Saville, member of the Beach Erosion Board, and includes the seven coastal engineering conferences held in 1950 through 1960, as well as the first conference on ships and waves in 1954 and the first conference on coastal engineering instruments in 1955.

These proceedings are not available from the Beach Erosion Board, but may be obtained as listed below (as available) from Council on Wave Research, Engineering Field Station, University of California, Richmond 4, California.

1st Conference on Coastal Engineering	- 1950	$4.50
2nd Conference on Coastal Engineering	- 1951	Out of print
3rd Conference on Coastal Engineering	- 1952	Out of print
4th Conference on Coastal Engineering	- 1953	Out of print
5th Conference on Coastal Engineering	- 1954	Out of print
6th Conference on Coastal Engineering	- 1958	$10.00
7th Conference on Coastal Engineering (2 Volumes)	- 1960	12.50
1st Conference on Ships and Waves	- 1954	6.00
1st Conference on Coastal Engineering Instruments	- 1955	5.00

At least some of these proceedings (including those out of print) are available on microfilm through the Council on Wave Research. Questions regarding availability of microfilm should be directed to the Council.

INDEX TO THE PROCEEDINGS OF THE COASTAL ENGINEERING CONFERENCES

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FIRST CONFERENCE ON COASTAL ENGINEERING INSTRUMENTS - 1955

FIRST CONFERENCE ON
COASTAL ENGINEERING INSTRUMENTS - 1955
(cont.)

FIRST CONFERENCE ON
COASTAL ENGINEERING INSTRUMENTS - 1955
(cont.)

PROGRESS REPORTS ON RESEARCH SPONSORED BY THE BEACH EROSION BOARD

Compiled by Thorndike Saville, Jr., Research Division
Beach Erosion Board

Summaries of progress made during fiscal year 1962 (i.e. to June 30, 1962) on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements as to the status of some research projects being prosecuted in the laboratory of the Beach Erosion Board, are presented below. These summaries supplement and continue those contained in prior issues of the Bulletin.

I. University of California, Contract DA-49-055-eng-8. Sources of Beach Sand.

Seasonal sampling of the eighteen beaches in the San Francisco area was continued during the early part of the year. A report "Beaches Near San Francisco, California, 1957-1958" describing some of the observations on these beaches was given limited distribution by the University of California as Institute of Engineering Research Technical Report, Series 14, Issue 23. This report indicated that beach profiles and materials differed considerably from beach to beach on the individual beaches observed, and from time to time at the same beach. However, the data apparently neither supported nor disproved the hypothesis that beach sand is statistically finer and better sorted in summer than in winter. The report concludes that the long intervals between sampling dates may have been responsible for many of the inconsistencies observed during the 1957 to 1958 period and that it is possible that when beaches are occupied at relatively long intervals of time, beach material and profile parameters may become primarily functions of the vagaries of wave energy just prior to the sampling, rather than functions of seasonal trends.

Sampling of the beaches north of the Russian River was discontinued as it was felt that the prime purpose of the investigation (i.e. to determine the general distribution of grain size and foreshore slope on these beaches at different seasons) had been accomplished. A report "Beaches in Northwestern California" describing these observations was prepared and given limited distribution by the University of California as Institute of Engineering Research Technical Report, Series 14, Issue 25. This report describes conditions at the various beaches sampled, and indicates differences between them. The report indicates that although groups of stations having sands that are not significantly different, usually fall within boundaries of geographic units, the sands at any given locality can seldom be correlated with the existing wave exposure; consequently it is concluded that beach character must reflect other causes as well, such as material source, location, and quantity.

In addition, data obtained partially in conjunction with this contract, on the use of naturally radioactive thorium in beach sands as a means of detecting the direction of littoral drift, was used in a report "Transportation of Coastal Sediments" and given limited distribution by the University of California as Institute of Engineering Research Technical Report, Series 185, Issue 1. It is being published as Technical Memorandum No. 131 of the Beach Erosion Board under the title "Littoral Studies Near San Francisco Using Tracer Techniques".

II. Massachusetts Institute of Technology, Contract DA-49-055-eng-16. Sorting of Beach Sand by Waves.

Work has proceeded in both an experimental and a theoretical direction toward the determination of dissipation of wave energy in the breaker region and the generation of longshore currents. The experimental program has been primarily concerned with the development of an adequate orbital velocity probe, utilizing a thermistor. The theoretical program has proceeded toward obtaining equations for the orbital velocities and particle displacements in very shallow water and the breaker region, initially using the work of Biesel. His development has been extended to higher order terms in wave amplitude in order to introduce the effect of mass transport. It is planned to program the resulting formulae on a computer to yield the appropriate wave parameters as functions of depth, distance, and beach slope.

III. University of California Contract DA-49-055-eng-17. Fundamental Mechanics of Sand Movement by Waves.

A report "A General Reconnaissance of Coastal Dunes in California" was completed and published as Miscellaneous Paper No. 1-62 of the Beach Erosion Board. This report describes the dune localities along the California Coast, and examines their common features. Dune forms, beach configuration and condition, activity of dune sand, and sediment sources are the factors considered in describing the localities. Experiments were conducted in a 4x4x100-foot wind channel in which the performance of the portable sand trap developed previously was compared with the total transport obtained by a trap at the end of the channel.

Further theoretical work has been carried out on sediment transport by wave action using, initially, a statistical approach. The two parameters describing flow intensity and sediment rate respectively were determined independently, but their functional relationship include constants which can be determined only by experiment. Data obtained previously by Manohar have been explored for this determination, but indicate that additional experimental data are required.

IV. University of California, Contract DA-49-055-eng-44. Laboratory Study Of Wave Refraction

Additional work was done in the ripple tank on the Mach stem reflection phenomenon, using concave barriers with wavy and roughened surfaces.

A report "Solitary Wave Behavior at Concave Barriers" was prepared and given limited distribution by the University of California as Institute of Engineering Research Technical Report, Series 89, Issue 7. This report describes the behavior of a solitary wave as it advances along concave barriers for the case of an initial angle of 12° between the barrier and the direction of incident wave advance. For concave vertical barriers it was found that the wave height at the barrier increased continuously in the direction of wave travel. The condition of the surface of the barrier did not affect this build-up. The wave did not reflect from the barrier, but instead the wave crest bent next to the barrier so that it became virtually perpendicular to it. The highest build-up at the barrier was approximately 2.5 times the height of the incident wave. On the other hand, as the solitary wave advanced along a sloping rubble mound barrier, this build-up of the wave next to the barrier never occurred. However, there was still no reflection.

Some preliminary work was done on programming wave diffraction theory for an IBM 704 computer. Some rather small scale model tests were iniated to study the effect of bottom slope shoreward of a breakwater on wave characteristics in the protected area (that is, the combined effect of refraction and diffraction).

V. Dr. W. C. Krumbein (Consultant). Study of Beach Sampling Methods.

A report "The Analysis of Observational Data from Natural Beaches" has been published as Technical Memorandum No. 130 of the Beach Erosion Board, and summarizes work done on the application of computing machine methods to the study of factors influencing beach characteristics and stability. The report indicates that design of field tests to acquire large and complex data must be carefully made, but points out that when such tests are properly designed, there is a high probability that individual sources of variability may be identified and evaluated while in the presence of other naturally occurring but changing variables. The interplay or interlocking of several variables may also be evaluated by the methods derived. Further study, utilizing computer techniques, is being initiated using sand sample data collected near the mouth of the Cape Fear River, North Carolina.

VI. Beach Erosion Board Laboratory.

(a) Wave Forces on Structures.

Analysis was continued on the wave force data obtained in the large wave tank on a vertical 12-inch diameter pile. A report was prepared for presentation at the American Society of Civil Engineers Houston meeting in February 1962; it presents the continuous variation of simultaneously occurring wave force, water velocity and acceleration, and wave profile through one or more wave cycles for a limited number of wave conditions.

(b) Rubble Mound Stability.

Large scale tests on stability of rubble mound structures under wave action were continued to spotcheck the results of the small scale test program at the Waterways Experiment Station. Calibration of the tank to obtain a precise determination of the wave heights at the structure location without the structure in place were completed for the waves tested on the 1 on 1-1/2 slope rubble breakwater, having cap protection of approximately 1-foot diameter, 160-pound stone. The results of these tests appear to indicate that the effects of scale, at least insofar as stability is concerned, are small and essentially indistinguishable from effects caused by differences in stone shape, placement, and degree of interlocking. Tests have been initiated using a 75-pound, 4-legged concrete shape (a quadripod) as the cap protection. This shape is somewhat similar to a tetrapod with a flat base. Tests to determine the no-damage (or incipient damage) condition have been completed for several wave periods. Tests involving larger, damaging waves will be carried out in the spring of 1963.

(c) Wave Run-up.

Some large scale run-up data involving waves up to 3.5 feet in height were gathered on a 1 on 1-1/2 rubble (quadripod) breakwater in conjunction with tests on rubble mound stability. Additional run-up data involving waves up to about 6 feet in height were gathered on smooth sand beaches having a 1 on 15 slope in conjunction with beach deformation tests in the large wave tank.

In addition to correction curves for scale effect in wave run-up published in the revised edition of Beach Erosion Board Technical Report No. 4, a generalized run-up curve embodying an average scale correction was included in a report "Freeboard Allowances for Waves in Inland Reservoirs" and published in the American Society of Civil Engineers Waterways and Harbors Division Journal.

(d) Study of Sand Bypassing Operations.

Efforts were continued to collect all available data on sand bypassing operations (past, present, or planned) for correlation and study. Analysis of the hydrographic survey data obtained in the Port Hueneme area in June 1959 is essentially complete. Comparison of this data with previous survey data taken since the bypass dredging operation in 1954-55 reveals that accretion north of the jetties slowed after the filling of the dredged pocket and that the downcoast (southern) beaches continued to erode at an average rate of approximately 1,200,000 cubic yards per year.

The field observation program was continued in the vicinity of Ventura County Harbor, California, in which an offshore breakwater (parallel to the shore) forms a protected area serving as a sand trap. Use of 3 wave gages at this location, each with different degrees of sheltering, permits some

degree of evaluation of wave direction - at least to the extent of permitting separation of those waves which are from the southerly quadrant from those which are from the northwesterly quadrant. Coordination with Palm Beach County officials was continued in compiling data and information from the operation of the sand transfer plant at Lake Worth Inlet, Florida.

A report "Mechanical Bypassing of Littoral Drifts at Inlets" was prepared and published in the Waterways and Harbors Division Journal of the American Society of Civil Engineers. It presents a brief examination of the general processes of littoral drift movement at both uncontrolled and controlled coastal inlets. The report indicates the principal factors to be evaluated when mechanical bypassing of littoral drift past an inlet is considered, and examines the general techniques of bypassing. The report also presents a summary of all completed and active bypassing projects in the United States.

Specifications for the purchase of a mass-flow density meter (utilizing a radioactive source) to measure flow of sand through a 3-inch pipe were completed, and submitted for bids. The meter will be tested in conjunction with model work on littoral drift rates at the Board's laboratory and evaluated as to feasibility for use in the field to measure quantity of material pumped in bypassing operations.

(e) Laboratory Study on Relation of the Littoral Drift Rate to Incident Waves.

A report "Laboratory Determination of Littoral Transport Rates" was published in the Journal of the Waterways and Harbors Division, American Society of Civil Engineers. This report discusses previous littoral drift studies, adds data obtained at the Beach Erosion Board, and presents a figure relating the value of alongshore littoral transport to the value of the alongshore component of wave energy summed over the period of interest. The report also discusses techniques of laboratory studies of the littoral transport rate, indicating some problems and solutions for measurement, feeding and trapping mechanisms, end effects caused by refraction at the ends of the test beach, and wave variability (perhaps superimposed basin surges) over the test area. A series of additional laboratory tests were made in the Shore Processes Test Basin to obtain further data on the relation of littoral movement to incident wave characteristics. One of these involves a maximum movement rate of about 19,000 to 20,000 pounds of sand per hour (dry weight) obtained with the highest wave tested of about 11 inches. This rate of movement is the equivalent of about 60,000 cubic yards per year. Testing was also continued on the effect of local variation in wave period on the rate of littoral movement. In general, it was found that varying the wave period over a short range about the mean period (plus or minus 10%) at lesser and lesser time intervals increases the rate of littoral movement. However, a single test in which the wave period was varied continuously (sinusoidaly) over a one-minute cycle showed a decrease in transport rate from the test in which the period was varied

abruptly in one-minute intervals. An energy comparison shows that the continuously changing wave delivers about 1.5% more energy than the waves of the tests having a fixed time interval between wave period changes. This result is therefore inconsistent with the two general findings of more energy - more transport and decreasing time intervals - more transport, and consequently cannot be explained in terms of the quantity of energy delivered by the waves or the time interval of wave variability. Since most laboratory tests have been run with a single constant wave period, this general finding has considerable implication in the interpretation of earlier results. This variation in wave period is, of course, more nearly a modeling of actual wave conditions, although still greatly different from the true wave spectrum generated in nature.

(f) Measurement of Suspended Material in Laboratory Wave Tanks.

Additional suspended sediment samples were obtained (under wave action) utilizing a pump-type sampler in a wave flume testing lower specific gravity coal rather than sand. It is hoped that these measurements may aid in defining scale relations and prototype measurements. Comparison is being made with measurements of sand in suspension under the same wave conditions, and with waves scaled upward according to the settling velocity of the sediment. A few suspended sediment samples were also taken in the large wave tank with waves of 2 to 5 feet in height in conjunction with tests on beach deformation.

(g) Equilibrium profile, beach deformation, and model scale effect studies.

Testing was continued in a small tank utilizing low specific gravity material (crushed coal) to study the effect of scale on movable bed models under wave action. The specific gravity of the coal was chosen so as to model the settling velocity of the sand used in earlier large scale tests. Profiles derived from these tests appear to bear basic resemblance to the profiles obtained with the large (5.5-foot) waves on a sand beach, particularly in the nearshore area shoreward of the breaker. Some of the differences observed may be attributable to the variation in specific gravity of individual coal grains , although the average specific gravity of the coal is as desired.

A series of beach deformation tests were initiated in the large wave tank, using waves of 2 to 6 feet in height acting on a 1 on 15 beach slope of 0.4-mm median diameter sand. The profiles resulting from these tests will be compared with tests made some years previously with 0.2-mm diameter sand. The immediate purpose of the tests was to enable estimates of the greater width of beach fill required for smaller grain sand sizes to provide protection equivalent to coarser sands, with application to requirements for emergency protection following the east coast storm of March 1962. However, the results should also serve the broader purpose of advancing our knowledge of beach deformation under wave action in general, and also of general scaling relationships. Because of the impracticability

of removing other test models from the large wave tank for these beach deformation tests, these tests were performed with a shorter tank length than were the corresponding tests with the 0.2-mm sand. It was recognized that this might introduce different reflection effects into development of the comparative profiles, but it was felt that these effects would not be large, particularly in the nearshore area of most interest. However, in an attempt to secure additional information on such possible reflection effects, the 0.2-mm tests which had been modeled using crushed coal are being repeated with the shorter (model) tank length. Repetition will enable comparison of test results with the coal for the two different tank lengths.

(h) Wave Measurements and Analysis

Wave gage records were taken at all field gage locations until March 5 when gages at Palm Beach and Atlantic City were destroyed by the severe east coast storm. Reinstallation is planned following rehabilitation and reconstruction of the piers at these locations. A gage installation at Virginia Beach, Virginia, is also planned in cooperation with local elements. A new type bottom pressure gage will be installed at Atlantic City, New Jersey, in addition to the normal staff gage. Comparison of the records from these gages both by direct time comparison and by comparison of the spectrum analyses, should bring about better knowledge of the wave pressure attenuation with depth. Magnetic tape recorders to obtain tapes for analysis on the spectrum analyzer have been installed with virtually all gages. Additional tape recorded wave data have been obtained at many of the field locations, and further spectral analyses made on the analyzer. The actual shape of the height components at various selected periods in the spectrum have been extracted for certain wave conditions, and analysis thereof is underway. Further work with the magnetic tape analyzer has enabled quantitative values to be assigned to the wave heights associated with the various periods of the spectrum. A program for testing feasibility of using two staff resistance wave gages (relay type) for determining wave direction was carried out in the large wave tank, but preliminary results indicate that while the method worked fairly satisfactorily for long period waves, considerable difficulties ensue with short period waves.

(i) Regional Studies.

A report "Littoral Materials of the South Shore of Long Island, New York" has been published as Technical Memorandum No. 129 of the Beach Erosion Board. This report describes the physical characteristics of littoral materials along the south shore of Long Island, tabulating median diameter, and sorting and skewness coefficients for beach and bottom sediments for comparable zones of the profile and comparable survey periods at all locations where such data are available. In addition, a limited amount of data on physical properties such as mineral composition, roundness and sphericity of grains, specific gravity, and mass density are also presented. Additional compilations are underway on geomorphological and

littoral material data for the coastal sector from Cape Henlopen, Deleware, to Cape Charles, Virginia.

A comprehensive program has been initiated to develop data on offshore deposits of material along the Atlantic Coast that could be utilized in beach fill or nourishment projects. The study of the offshore deposits is intended to encompass all factors pertinent to the use of such deposits in the beach zone such as location and magnitude of deposit, characteristics of material composing deposit, type of plant needed and economics of transferring material from the deposit zone to points onshore.

(j) Technical Report No. 4, "Shore Protection Planning and Design"

The revised edition of Technical Report No. 4 has been published and is available for purchase from the Superintendent of Documents, Government Printing Office, Washington 25, D. C. at $3.00 per copy in the United States. (Foreign orders should include $0.75 additional to cover postage). Major revisions and additions involve sections on wave run-up, hurricane waves and surge, design criteria for rubble mound stability, and wave forces on piles. A continuing study is made to improve present chapters of this publication by the obtention, analysis, and review of data and reports on coastal engineering which may be appropriate for inclusion as having application to practical engineering problems.

(k) Re-examination of Beach Protection Projects.

A continuing program is being carried out on the re-examination of beach fill and periodic nourishment projects to determine the effectiveness of the fill material within the beach zones, and to better establish the factors upon which the desired characteristics of fill material are based. Continuing studies of other projects constructed following Beach Erosion Control studies are also underway to determine effectiveness of the various structure components. In particular data have been obtained on completed projects at Sea Side Park, Connecticut, and Key West, Florida; sand sample data have also been collected near the mouth of the Cape Fear River, North Carolina, in conjunction with this work.

Over the past 25 years, many ground level photographs have been taken of beaches on the New Jersey coast. Compilation of this photographic data has been underway and a pictorial history type report is nearing completion. Many of the photographic locations have been re-established, and comparative photos obtained during this year.

(l) Experimental Studies, Effectiveness of Sand Fences.

In cooperation with the State of North Carolina and the Wilmington District of the Corps of Engineers, a study has been underway on Core Bank, one of the Outer Banks of North Carolina, on the effectiveness of various types of sand fence in building and stabilizing dunes. A series

of 1000-foot sections of sand fence of various types and arrangements were installed on Core Banks, and periodic examinations and surveys made to obtain information on the comparative effectiveness of various sections. A self-contained wind measuring instrument has been installed in the test area to obtain information on wind velocity during the experiment. It has operated more or less continuously, although difficulties have been experienced from time to time with the take-up mechanism for the recorder paper, the inking mechanism, and the power supply. An interim report covering the first year of the experiment has been prepared and was summarized in a report given at the May meeting of the American Shore and Beach Preservation Association. This report is to be published by the American Shore and Beach Preservation Association in their forthcoming October issue of "Shore and Beach". A further and more detailed report is being prepared for the Eighth Conference on Coastal Engineering in the fall of 1962.

Two specific conclusions from the data are that (at least for this area and local conditions) straight sand fencing trapped and held as much sand as either a zigzag or side-spur fence configuration, and that fencing installed initially on a bull-dozed 2-foot sand ridge does not trap sand as well as fencing installed initially on normal level ground. The sand accumulation of the fences was flattened by the March storm, and some of the fences have been destroyed by overwash of the island during portions of the year. It is planned that new fencing, as required, will be installed by the State in November 1962, after the close of the hurricane season. In connection with this project also, some experiments have been made to determine the feasibility of a siphon-type tide gage and a bubble-type tide gage for use in both the sound and ocean areas.

(m) Model Determination, Scour in Front of Seawalls

The wave basin study attempting to experimentally relate depth and lateral extent of scour at toes of seawalls to incident wave and beach sediment characteristics has been continued. Additional data were obtained with a 0.4-mm median diameter sand for comparison with earlier tests with 0.2-mm sand. Analysis of these data is now underway, and contour maps showing elevations in the vicinity of the seawall are being analyzed. Although the program has involved so far only a vertical wall located at the still water level, other types of walls and locations relative to still water level may be tested in the future.

(n) March 1962 Storm Study

A comprehensive program involving data collection and general field observations, was initiated to study the effects of the March 1962 east coast storm. Several teams were sent to the field immediately following the storm to obtain data on the effects of the storm on the beach backshore zones and on shore protective structures, as well as other data pertinent to these zones. Virtually the entire shoreline between Montauk Point, Long

Island, New York, and the North Carolina-Virginia state line was covered. Aerial photographs of the coast immediately following the storm were obtained, and it is planned to repeat these at later intervals in the year for comparative purposes. Further field data will be collected to evaluate the emergency repair work which has been carried out since the storm.

(o) Evaluation of a Stream Outlet Structure (Sand Plug Opening Device)

A laboratory study was made for, and a report prepared and submitted to, the U. S. Soil Conservation Service on the effectiveness of a sand plug opening device. The device was designed by Soil Conservation Service Engineers in an attempt to have stream floods break through wave formed sand plugs in the seaward exits of streams before the stream floods reach an elevation high enough to cause damage inland of the sand plug. In general, the stream flood cannot be allowed to rise high enough to overtop the sand plug.

A 1:20 scale model of the proposed structure was made and tested in a laboratory wave tank. The sand plug opener worked as predicted; however, the study left some doubt as to whether the device would cause the sand plug to erode quickly enough to prevent flooding when the stream flood rose from +2 feet MSL to +6 feet MSL in 13 minutes. Flooding was assumed to occur at stream elevations greater than +6 feet MSL.

VII Publications

Technical Memoranda published by the Board during fiscal year 1962 are listed below. Copies can be furnished on request to persons within the United States to the extent of a limited printing.

T. M. No.	Title and Date
126	Equilibrium Characteristics of Sand Beaches in the Offshore Zone, July 1961
127	Behavior of Beach Fill and Borrow Area at Prospect Beach, West Haven, Connecticut, August 1961
128	Geomorphology of the South Shore of Long Island, New York, September 1961
129	Littoral Materials of the South Shore of Long Island, New York, November 1961
130	The Analysis of Observational Data from Natural Beaches, November 1961

Material covered by the Technical Memoranda listed above is briefly described in the foregoing paragraphs (I to VI) Research Progress or in the section of Research Progress in volume 15, July 1961, of the Annual Bulletin of the Beach Erosion Board. Their contents are also briefly abstracted in the Annotated Listing of Beach Erosion Board Publications included at the end of this issue of the Annual Bulletin.

BEACH EROSION STUDIES

Beach erosion control studies of specific localities are usually made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2 of the River and Harbor Act approved 3 July 1930. By executive ruling the costs of these studies are divided equally between the United States and the cooperating agencies. Information concerning the initiation of a cooperative study may be obtained from any District or Division Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this Bulletin. Summaries of reports transmitted to Congress since the last issue of the Bulletin and lists of completed and authorized cooperative studies follow.

SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

BELLE PASS TO RACCOON POINT, LOUISIANA

The purpose of the investigation was to determine the best method of preservation or stabilization of the offshore islands comprising the study area. Two groups of low barrier beach islands about 1/2 mile wide separate Timbalier Bay, Terrebonne Bay, Lake Pelto and Caillou Bay from the Gulf of Mexico. The length from Belle Pass at the mainland end of the chain to Raccoon Point at the west end of the Isles Dernieres chain totals about 47 miles. The Timbalier chain is about 20 miles long and the Isles Dernieres-Wine Island chain about 24 miles long. The islands consist generally of sandy beaches backed by low dunes, thence low marshy areas. They are uninhabited and undeveloped except for well activities of the oil industry. The State owns East Timbalier Island and the western half of Timbalier Island. The remainder of the study area is privately owned.

Tides in the Gulf of Mexico in the area are diurnal, their mean range being 1.3 feet. Although specific storm tide data were not available, general historical information indicated that the islands have been completely inundated on several occasions. Over the period of record, the mainland near Belle Pass and the eastern end of East Timbalier Island have receded at a rate of about 100 feet per year. However, the north shore of the island is also moving toward the mainland, low sections are filling and the spit is extending westward. The gulf shore of the east end of Timbalier Island has receded at an average rate of 35 feet per year, but the shore of the western portion has advanced gulfward indicating a process of orienting Timbalier Island to the same alignment as East Timbalier Island. The west end of Timbalier Island has also extended westward into Cat Island Pass. The changes in these islands indicate a predominant westward littoral drift. Isles Dernieres have receded at an average rate of about 25 feet per year. They have extended westward, indicating a westward littoral drift, but have also extended eastward probably due to the tidal current effect of Wine Island Pass. These islands have generally deteriorated, indicating a lack of supply of material.

The District and Division Engineers concluded that the Isles Dernieres would probably continue to exist and provide protection for the mainland for a nominal period of 100 years and that the Timbalier chain will have a longer life expectancy; that the only plan of protection that would be suitable for both would be the replenishment of the beaches with an average annual deposition of about 1,200,000 cubic yards of material from the offshore Gulf area for each of the two island chains; and that the decision as to the degree of protection desired should be based on the economic benefits to local interests. They recommended that no project be adopted by the United States for stabilization and protection of the offshore islands in the area. The Beach Erosion Board concurred generally in their conclusions and recommendations. The Board concluded that a plan for completely stabilizing the island chains would involve raising the backbone ridge of the islands by artificial fill to minimize damage by overtopping, and artificial nourishment in sufficient quantity to stabilize the Gulf shores of the islands. A partial plan to retard the deterioration of the islands would comprise raising low sections of the backbone ridge or low areas of potential breaching by artificial placement of fill, and placing stockpiles of material on the Gulf shores to assist existing littoral drift in stabilizing those shores. The minimum measures which would be helpful in reducing deterioration would comprise placing any material dredged in connection with dredging of canals to reinforce the backbone ridge, encouraging mangrove growth by seeding or transplanting in the marshy bay sides of the islands to reduce losses by overtopping, and preventing any action in connection with oil drilling operations which would make the backbone ridge more vulnerable to overtopping or breaching. The Board found little public interest involved in the considered improvements and recommended that shore protection which may be undertaken by local interests, based upon their own determination of economic justification, be accomplished generally in accordance with the methods discussed in the report. The Board further recommended that no project be adopted by the United States at this time authorizing Federal participation in the costs of measures for the stabilization of the shores within the area covered by the report. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

VIRGINIA BEACH, VIRGINIA

The purpose of the study was to review the existing project and to determine the extent, if any, of Federal participation in the cost of periodic beach nourishment of the existing beach erosion control project, in accordance with provisions of Public Law 826, 84th Congress. Virginia Beach is located on the Atlantic Ocean in southeastern Virginia, about 3.5 miles south of the entrance to Chesapeake Bay, and 19 miles east of Norfolk, Virginia. It is a summer resort with a permanent population of about 8,400, but a maximum summer population of about 50,000. About 95 percent of the shore frontage is publicly owned.

The existing beach erosion control project for Virginia Beach comprising artificial placement of suitable sand fill to widen the beach berm to a minimum width of approximately 100 feet, and a system of groins as deferred construction when experience indicates the need thereof, was authorized by the River and Harbor Act of 1954. The beach fill was completed in 1953 with Federal aid as authorized. The project also contemplated periodic nourishment of the restored beaches at non-Federal expense, as the project was authorized prior to passage of Public Law 826, 84th Congress, which provides a policy of Federal assistance to periodic nourishment. Completion of the project has resulted in obtaining the benefits anticipated therefrom and has led local interests to embark on a program for improvement of resort facilities. The periodic nourishment program is a necessary part of the existing project, and will insure continuation of the benefits from prevention of damages to public property and recreational benefits from use of the publicly owned shores. The District and Division Engineers concluded that periodic nourishment is the most suitable and economical remedial measure to provide stability to the shore, and that the beach nourishment plan requiring placement of about 45,000 cubic yards of sand annually can thus be considered construction eligible for Federal participation in accordance with the provisions of Public Law 826, 84th Congress. The Beach Erosion Board recommended modification of the existing Federal project for Virginia Beach to authorize Federal participation by the contribution of Federal funds in the amount of one-third of the costs of periodic nourishment of the shore for a period of 25 years from the date of commencement of operations in placing an initial quantity of nourishment material equal to the deficiency in the design beach at that time, generally in accordance with the plan of the District and Division Engineers with such modifications thereof as in the discretion of the Chief of Engineers may be advisable. The Board further recommended continuation of the requirements for Federal participation in the existing project, but with the recommended Federal assistance in the costs of periodic nourishment. The Chief of Engineers concurred in the views and recommendations of the Board.

CAROLINA BEACH AND VICINITY, N. C.

The purpose of the investigation was to devise effective means of restoring an adequate recreational and protective beach and preventing further erosion of the shore. In addition to the single-purpose shore protection plan developed under the foregoing investigation, the report submitted by the District and Division Engineers included study of the needs and methods for protection against damages caused by hurricanes under the provisions of Public Law 71, 84th Congress, resulting in a dual-purpose plan which would provide both hurricane and shore protection. In its review of the report the Beach Erosion Board gave consideration to the technical adequacy of both plans, but limited its consideration of project justification and Federal participation to the single-purpose shore protection plan in accordance with its statutory function as prescribed in section 3 of Public Law 166, 79th Congress.

The study area is located in New Hanover County and comprises a 7-mile length of shore located about 15 miles south-southeast of Wilmington, North Carolina. The study area includes the Towns of Carolina Beach and Kure Beach and the unincorporated communities of Wilmington and Hanby Beaches. These communities comprise an important summer recreational area. The permanent population of New Hanover County, which includes the City of Wilmington, was over 71,000, according to the 1960 census. The summer population is increased by thousands of vacationists. Of the total shore frontage of 25,800 feet considered for protection, frontages about 1,200 feet in Carolina Beach and 1,900 feet in Kure Beach are publicly owned. The northern portion of the coastal area under study is a narrow barrier beach separating Myrtle Sound from the Atlantic Ocean. The remainder consists of a section of the peninsula mainland lying between Cape Fear River and the ocean. The shore of the study area is exposed to ocean waves with unlimited fetch. The predominance of wave energy components is from the northeast quadrant and produces a dominant southward littoral transport. The estimated annual deficiency in supply of material in the zone between the dune line and the 24-foot depth contour from 1938 to 1957 for the 5-mile reach including Carolina Beach and Kure Beach averaged about 79,000 cubic yards or about 3 cubic yards per linear foot of shore in the study area. The ocean mean tidal range is 4.2 feet. The highest ocean level of record, about 10.5 feet above mean sea level, occurred during hurricane "Hazel" in 1954. Two inlets into Myrtle Sound are located respectively 3 and 3.5 miles north of Carolina Beach. The more southerly one was artificially opened by local interests in 1952. The other inlet was opened by wave action during hurricane "Hazel" in 1954. No works have been provided to stabilize these inlets.

The District Engineer developed plans for protecting the shore of the study area against both erosion and hurricane damages, and concluded that the best protection against hurricane damage, commensurate with costs, comprises a dune with a crown width of 25 feet at an elevation of 15 feet above mean low water, a beach berm 50 feet wide at an elevation of 12 feet above mean low water, a feeder beach north of Carolina Beach and periodic nourishment as required. An alternative plan for shore protection alone, consisting of a beach fill to provide a berm 100 feet wide at an elevation of 8 feet above mean low water, a feeder beach and periodic nourishment, was also developed.

The District and Division Engineers made economic analyses of the foregoing plans of shore and hurricane protection for the entire study area (Carolina Beach to Kure Beach, inclusive), and separately for Carolina Beach only. They concluded that the dual-purpose plan of protection is amply justified by evaluated benefits. They found that public benefits justify Federal aid to first and periodic nourishment costs for shore protection under the provisions of Public Law 826, 84th Congress, and that prospective benefits justify Federal aid to hurricane protection under the policy established by previous authorizations of hurricane protection projects. Accordingly they recommended adoption of a dual-purpose Federal

project under which the United States would pay 59.7 percent of the first costs thereof and 11.6 percent of the annual costs for periodic nourishment for a period of 10 years.

The Beach Erosion Board concurred generally in the features of both the single-purpose and dual-purpose plans of protection for the shores of Carolina Beach and vicinity. The Board concluded that while it would be possible to provide effective erosion control independently of hurricane protection, provision of the latter by means of constructing and maintaining a berm and dune of adequate height and width necessitates effective supplemental measures for beach erosion control. It was the opinion of the Board that the dual-purpose plan is a technically practicable partial protection plan for the Carolina Beach and adjacent areas. Although it would provide reasonable protection under design storm conditions as contemplated by the reporting officers, it would provide little protection under conditions of the maximum hurricane that could reasonably be expected to occur in this area. The Board therefore considered that an adequate warning system, as well as plans and routes for rapid evacuation of the coastal region, would be essential supplements to the plan of protection recommended by the District and Division Engineers in order to prevent loss of life. Regarding the dual-purpose plan, the Board also pointed out that the berm elevation of 12 feet above mean low water is higher than the natural berms formed by wave action in this area, and that part of this high berm may be removed during storm profile adjustments. Such berm restoration as would occur during subsequent wave action would no doubt be at the lower natural level associated with the wave and tidal characteristics of the region. As the effectiveness of the hurricane protection is dependent on the high berm, the Board emphasized the importance of the program to reform the high berm annually before the hurricane season, as contemplated in the District Engineer's plan. Accordingly, subject to determination by the Chief of Engineers after review by the Board of Engineers for Rivers and Harbors that the dual-purpose plan for the entire area is suitable and economically justified, the Beach Erosion Board recommended adoption of a project for about 25,800 feet of the shores of Carolina Beach and vicinity to authorize Federal participation in the costs of a plan for protection of the shores from hurricane and erosion damage comprising for the latter feature alone artificail placement of fill to provide a beach berm 100 feet wide at an elevation of 8 feet above mean low water, a feeder beach, and periodic nourishment of the beach, the Federal aid for the latter to apply for a period of 10 years from the year of completion of the initial placement, substantially in accordance with the plan developed by the District and Division Engineers, with such modifications thereof as in the discretion of the Chief of Engineers may be advisable. The Board of Engineers for Rivers and Harbors and the Chief of Engineers concurred in the conclusions and recommendations of the Beach Erosion Board.

SHEFFIELD LAKE COMMUNITY PARK, OHIO

The purpose of the investigation was to determine the best means of protecting park property and of restoring and improving the beach to provide a public bathing beach. The study area, located in Sheffield Lake Village, Lorain County, Ohio, included the shore of the Community Park about 800 feet in length, all owned by the village. The 1960 permanent populations of the village and Lorain County were respectively 6,884 and 217,500. The shore of the study area is exposed to waves of Lake Erie. The predominance of energy components in deep water is such as to produce a slight predominance in westward littoral transport. However, due to the resistant cliffs adjacent to the study area and protective measures, little material is available to the beaches. The mean level of Lake Erie is about 2 feet above low water datum and the highest monthly mean stage is 4.2 feet.

The District Engineer developed a plan for restoring a protective and recreational beach along the park frontage. He concluded, and the Division Engineer and Beach Erosion Board concurred, that the most practicable plan of improvement consists of placement of sand fill, and construction of two groins. The District Engineer made an economic analysis of that plan of protection.

The District and Division Engineers and the Beach Erosion Board concluded that the plan of protection and improvement is economically justified. They found that the shore is publicly owned and that, based on the provisions of Public Law 826, 84th Congress, public benefits justify Federal aid of one-third of the first costs. Accordingly, they recommended adoption of a Federal project to provide for Federal participation, subject to certain conditions, to the extent of one-third of the first costs of protecting about 800 feet of the park shore by widening the beach to a minimum berm width of 40 feet at an elevation of 8 feet above low water datum, and construction of two groins. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

STATE OF NEW HAMPSHIRE

The purpose of the investigation was to develop plans for restoration and stabilization of adequate recreational and protective beaches, and for protection of bluffs, headlands and coastal roads, also to review the existing Federal project for Hampton Beach. The study area comprised the entire ocean shore of New Hampshire between the entrance to Portsmouth Harbor and the Massachusetts State line. It included the shores of the Towns of New Castle, Rye, North Hampton, Hampton and Seabrook, a total shore frontage of about 18 miles. The shore area is developed principally for summer recreational use. In 1957 the permanent population of the coastal towns was about 11,200. The summer population of these towns is about 60,000, but over weekends increases to nearly 100,000. The coastal State highway closely follows the irregular shore and much of the shore frontage

is within the State's right-of-way. The shore area of Hampton Beach, the principal resort community, is owned by the State and the Town of Hampton. Although the shores of the study area are fully exposed to waves of the Atlantic Ocean approaching from the northeast and east, there is little evidence of a predominant direction of littoral drift, except at Hampton Beach where southward transport is predominant. Tides are semi-diurnal, the mean range decreasing from 8.7 feet at Portsmouth to 8.3 feet at Hampton. The maximum tides are estimated at about 12 to 12.5 feet above mean low water, with tides of 3 feet or more above mean high water occurring about once in two years. The study area is characterized by headlands of unconsolidated glacial material which have supplied material to the beaches. Depletion of available material has reduced the supply and the beaches have gradually deteriorated. The building and maintenance of adequate beaches may be accomplished by artificial placement of sand. The rate of loss of fill can be reduced by groins.

The Division Engineer concluded that practicable plans which merit consideration for the protection and improvement of shores within the study area are as follows:

a. Seabrook Beach, Seabrook. - Restoring and protecting approximately 3,000 feet of beach by widening to a 150-foot width by direct placement of sand fill and enlargement and extension of the existing south jetty at the Hampton Harbor Inlet;

b. Hampton Beach, Hampton. - Construction of an impermeable groin 235 feet long;

c. Great Boars Head, Hampton. - Placement of riprap revetment around the toe of approximately 800 feet of the outer end of Great Boars Head;

d. North Beach, Hampton. - Placement of riprap revetment along the toe of approximately 2,000 feet of the steel bulkhead;

e. North Hampton Beach, North Hampton. - Restoring, protecting and improving approximately 1,600 feet of beach by widening to a 150-foot width by direct placement of sand fill and construction of an impremeable groin 460 feet long;

f. Little Boars Head (Vicinity of Fish Houses), North Hampton. - Construction of a stone mound approximately 330 feet long;

g. Little Boars Head to Juniper Point, North Hampton. - Construction of a stone mound and placement of riprap revetment along the toe of approximately 500 feet of bluff;

h. Bass beach, North Hampton. - Construction of a stone mound approximately 1,150 feet long;

i. Rye Beach, Rye. - Construction of a concrete-encased steel sheet pile bulkhead approximately 800 feet long south from South Street, and placement of riprap revetment along its toe, if needed, thence a mortared stone wall approximately 1,600 feet long and placement of riprap revetment along its toe;

j. Jenness Beach, Rye. - Construction of a concrete-encased steel sheet pile bulkhead and placement of riprap revetment along approximately 500 feet of public beach in the vicinity of Perkins Road;

k. Ragged Neck Point (Rye Harbor State Park), Rye. - Construction of a mortared stone wall approximately 2,000 feet long and placement of riprap revetment fronting 1,400 feet of the wall outside the shelter provided by the Rye Harbor north jetty;

l. Foss Beach (south end), Rye.- Construction of a steel sheet pile bulkhead approximately 200 feet long and placement of riprap revetment along its toe, if needed;

m. Foss Beach (north end), Rye. - Construction of a mortared stone wall approximately 1,150 feet long and placement of riprap revetment or a stone apron along its toe, if needed;

n. Rye North Beach, Rye. - Construction of a mortared stone wall approximately 1,850 feet long and placement of riprap revetment or a stone apron along its toe;

o. Wallis Sands Beach (entire), Rye. - Restoring, protecting and improving approximately 4,500 feet of beach by widening to a 150-foot width by direct placement of sand fill and construction of an impermeable groin 360 feet long;

p. Wallis Sands State Beach, Rye. - Restoring, protecting and improving approximately 800 feet of beach by widening to a 150-foot width by direct placement of sand fill and construction of an impermeable groin 350 feet long. This plan is an alternative to that described in subparagraph o. If all of Wallis Sands Beach is widened, the groin at the south end of the State beach would be unnecessary;

q. Seal Rocks to Pulpit Rock, Rye. - Construction of a stone mound approximately 400 feet long; and

r. New Castle Town Beach, New Castle. - Restoring, protecting and improving approximately 800 feet of town beach by widening to a 150-foot width by direct placement of sand fill and incidental widening of adjacent shore areas.

The practicable plans of protection, which the Division Engineer considered to have sufficient public interest and economic justification to

warrant Federal aid, are as follow: Hampton Beach, North Hampton Beach, and Wallis Sands Beach.

The Beach Erosion Board concurred in the conclusions of the Division Engineer and recommended that projects be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the costs of measures for the restoration and protection of the publicly owned shores at North Hampton and Wallis Sands Beaches, New Hampshire, substantially in accordance with the following plans of the Division Engineer, with such modifications thereof as may be considered advisable by the Chief of Engineers:

a. North Hampton Beach. Widening approximately 1,600 feet of beach to a 150-foot width by direct placement of suitable sand fill and construction of an impremeable groin about 460 feet long;

b. Wallis Sands Beach. Widening approximately 800 feet of the beach to a 150-foot width by direct placement of suitable sand fill and construction of an impermeable groin about 350 feet long.

The Board further recommended modification of the existing Federal project for Hampton Beach to authorize Federal participation in amount of one-third of the costs of constructing an impermeable groin about 235 feet long and one-third of the costs of periodic nourishment of the beach for an initial period of 10 years from the year of the first nourishment operation. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

RARITAN AND SANDY HOOK BAYS, NEW JERSEY

The purpose of the investigation was to develop the most suitable plans for restoration of the shore and protection of property against erosion along the bays' shores. In addition to single-purpose shore protection plans developed under the foregoing purpose, the report submitted by the District and Division Engineers included study of the needs and methods for protection against damages caused by hurricanes under the provisions of Public Law 71, 84th Congress, resulting in a dual-purpose plan which would provide both hurricane and shore protection. In its review of the report the Beach Erosion Board gave consideration to the technical adequacy of both plans, but limited its consideration of project justification and Federal participation to the single-purpose shore protection plans in accordance with its statutory functions as prescribed in section 3 of Public Law 166, 79th Congress.

The study area, which lies in Middlesex and Monmouth Counties, comprised the 21-mile length of the shores of Raritan and Sandy Hook Bays between South Amboy and Highlands. The western end of the study area is about 30 miles by highway southwest of midtown New York City. This shore is an important summer recreational area. The permanent population of

communities in the study area is over 142,000. The population is greatly increased by summer vacationists. About 23 percent of the shore in the study area is publicly owned. The coastal area under study includes high bluffs near the east and west ends and low marshlands in the intervening area. Beaches generally are narrow. The bluffs supply a limited quantity of beach material and a deficiency of supply results in a slow deterioration of the protective and recreational beaches. The shore of the study area is generally protected from waves of the Atlantic Ocean by Sandy Hook. The predominance of energy components is such as to produce a dominant westward littoral transport of beach material except near the east end of the area, but the rate of transport is generally low. The mean tidal range increases from 3.8 feet at Highlands to 5.0 feet at South Amboy. The highest estimated bay level, about 10 feet above sea level, occurred during hurricane "Donna" in 1960.

The District and Division Engineers developed plans for protecting the shore of the study area against both erosion and hurricane damages. The dual-purpose plan comprises beach fills and levees with a top elevation of 15 feet above mean sea level, and in the case of Keansburg the plan includes three groins. Alternative plans for shore protection (erosion control) alone consisting of placing beach fill to provide a berm 150 feet wide at an elevation of 5.5 feet above mean sea level, or 50 feet wide at an elevation of 10 feet in front of bluffs, were also developed. Maintenance of the stability of the shore would be accomplished by periodic replenishment of sand losses under either plan. The District and Division Engineers made economic analyses of the foregoing plans of shore and hurricane protection, and concluded that the hurricane and shore protection plans for Madison Township, and Keansburg and East Keansburg are amply justified by evaluated benefits, also that shore protection alone is justified for Matawan Township and Union Beach. They found that public benefits justify Federal aid to first costs for shore protection under the provisions of Public Law 826, 84th Congress, and that prospective benefits justify Federal aid to hurricane protection under the policy requiring 30 percent local cooperation as approved for hurricane protection projects in the Flood Control Act of 1958. Accordingly they recommended adoption of a project by the United States for the foregoing protection, the United States paying 64.8 percent of the first costs thereof.

The Beach Erosion Board concurred generally in their views that the plans of protection for the shores of the study area are practical plans for their respective purposes. However, it noted that the hurricane protection is designed for storm surges equivalent to those of record, but not for those of the maximum probable or even the standard project hurricane which might occur infrequently. Accordingly, the Board stated that it is imperative that local interests recognize that the plan would provide only partial hurricane protection. The Board considered that the single-purpose shore protection plans consisting of widening the beach to provide a berm 150 feet wide at an elevation of 5.5 feet above mean sea level, or 50 feet wide at an elevation of 10 feet above mean sea level in front of bluffs, are somewhat more than

required in this locality, and that a berm width of 100 feet at an elevation of 5.5 feet, or 25 feet at an elevation of 10 feet in front of bluffs, would be adequate. The Board also believed that groins may be needed at Keansburg for shore protection as well as for hurricane protection, and included them in the single-purpose shore protection plan for deferred construction when experience indicates their justification.

The Beach Erosion Board recommended adoption of projects for the shores of Madison and Matawan Townships, Union Beach and Keansburg, New Jersey to authorize Federal participation in the costs of plans for protection of the shores, comprising artificial placement of beach fill to provide for a beach berm 100 feet wide at an elevation of 5.5 feet above mean sea level, or 25 feet wide at an elevation of 10 feet above mean sea level in front of bluffs, and three groins at Keansburg as deferred construction when experience indicates the need thereof, with such modifications thereof as may be considered advisable by the Chief of Engineers. The foregoing recommended plans may be constructed separately or as a part of dual-purpose plans for Madison Township and Keansburg recommended by the Board of Engineers for Rivers and Harbors, if subsequently authorized. Federal assistance in the single-purpose shore protection plans would entail contribution of funds presently estimated to amount to 28.2 percent of the initial construction costs of the beach widening in the four sections indicated. The Board of Engineers for Rivers and Harbors and the Chief of Engineers concurred in the conclusions and recommendations of the Beach Erosion Board. The Chief of Engineers recommended dual-purpose plans, substantially as developed by the District Engineer, for Madison Township and Keansburg, and single-purpose shore protection plans at Matawan Township and Union Beach.

STATE OF CALIFORNIA, INTERIM REPORT ON VENTURA AREA

The purposes of the investigation were to determine the most effective and economical means of preventing further erosion of the shore line between the Ventura and Santa Clara Rivers, and specifically to reexamine the findings regarding the Pierpont Bay area from a previous report. The study area between the Ventura and Santa Clara Rivers is about 4 miles in length. It includes San Buenaventura Beach State Park with a frontage of about 2.3 miles, about 0.9 mile of which is a narrow strip of park area fronting private residential development in the Pierpont area. An existing beach erosion control project authorized in 1954 for the State Park provides for three groins to protect the main or developed portion of the park. These groins were not built at the time of the study.

The tides in the area are diurnal, the diurnal range being 5.3 feet and the mean range 3.7 feet. The principal wave action affecting the area is from the west and west-northwest. The predominant direction of littoral transport is downcoast (southeastward). A relatively constant supply of material arrives at the problem area by littoral movement along the shore

from northwest of Ventura River. Additional large quantities of material supplied at irregular intervals during floods by the Ventura and Santa Clara Rivers have caused accretion to the shore line in the Ventura problem area (due partly to the groin effect of the Santa Clara River delta), but construction of reservoirs in the drainage areas of these rivers and below-normal rainfall during the past 12 years have reduced the supply by at least one-half. This reduction in material supply, part of which will be permanent, has caused substantial recession of the shore line in recent years.

The District and Division Engineers concluded that the plan of improvement under the existing project is inadequate to provide protection required under existing conditions, and they developed a plan for protecting the Ventura and Pierpont Bay frontage comprising beach fill and nine groins. They found that benefits from prevention of loss of public and private lands and improvements, and recreational benefits justify the work.

The Beach Erosion Board concurred generally in their conclusions and recommendations and stated the following opinions. Changed conditions since adoption of the original project for this area in 1954 have resulted in expansion of the problem area and necessity for modification of the existing plan of protection. Construction of the groin system should begin with groin 1 at the downdrift (south) end of the system and continue northwestward. If groins 1, 2 and 4 comprise the initial construction, as currently proposed, their effectiveness should be observed as a basis for design of the remainder of the groin system. A lesser number of groins than the nine recommended may possibly be more economical for protection of the Ventura and Pierpont area than indicated by the District Engineer's present estimates. As the plan contemplates artificially filling the groins with sand to the limit of their impounding capacities, they will not interfere with passage of the available volume of littoral drift. Therefore no important adverse effect on downcoast shores is to be expected from this plan of protection, and no need for continuation of the groin system farther downcoast is apparent at this time.

The Beach Erosion Board recommended that, in lieu of the existing project, a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in an amount equal to one-third of the first costs of construction of nine groins and artificial placement of beach fill for the protection of approximately 2 miles of the shore at Ventura, California, substantially in accordance with the plan developed by the District Engineer, with such modifications thereof as may be considered advisable by the Chief of Engineers. The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

COMPLETED COOPERATIVE BEACH EROSION STUDIES

Location	BEB Report Completed	Published in H. Doc.	Cong.	Federal Projects Recommendation *	Federal Projects Authorized by Congress
ALABAMA					
Perdido Pass (Alabama Pt.)	18 Jun 54	274	84	Unfav.	
CALIFORNIA					
Santa Barbara - Initial	15 Jan 38	552	75	Unfav.*	
Suppl.	18 Feb 42				
Final	22 May 47	761	80	Unfav.	
Ballona Creek & San Gabriel R. (Partial)	11 May 38			Unfav.*	
Orange County	10 Jan 40	637	76	Unfav.*	
Coronado Beach	4 Apr 41	636	77	Unfav.*	
Long Beach	3 Apr 42			Unfav.*	
Mission Beach	4 Nov 42			Unfav.*	
Pt. Mugu to San Pedro BW	27 Jun 51	277	83	Fav.	3 Sep 54
Carpinteria to Pt. Mugu	4 Oct 51	29	83	Fav.	3 Sep 54
Oceanside, Ocean Beach, Imperial Beach & Coronado, San Diego County	26 Jul 55	399	84	Fav.	3 Jul 58
Santa Cruz County	13 Sep 56	179	85	Fav.	3 Jul 58
Humboldt Bay (Buhne Pt.)	29 Mar 57	282	85	Fav.	3 Jul 58
Newport Bay to San Mateo Creek, Orange County	3 Dec 59	398	86	Fav.	14 Jul 60
San Diego County	30 Jun 60	456	86	Fav.	29 Mar 61
Ventura	28 Dec 61	458	87	Fav.	
San Gabriel River to Newport Bay, Orange Co.	20 Apr 62	602	87	Fav.	
CONNECTICUT					
Compo Beach, Westport	18 Apr 35	239	74	Unfav.*	
Hawk's Nest Beach, Old Lyme	21 Jun 39			Unfav.*	
Ash Crk. to Saugatuck R.	29 Apr 49	454	81	Fav.	17 May 50
Hammonasset R. to East R.	29 Apr 49	474	81	Fav.	3 Sep 54
New Haven Hbr. to Housatonic R.	29 Jun 51	203	83	Fav.	3 Sep 54

* No established policy for Federal participation in construction of shore protection works existed prior to 1946.

Location	BEB Report Completed	Published in H. Doc.	Cong.	Recommendation *	Federal Projects Authorized by Congress
CONNECTICUT (Cont.)					
Conn. R. to Hammonasset R.	28 Dec 51	514	82	Unfav.	
Pawcatuck R. to Thames R.	31 Mar 52	31	83	Unfav.	
Niantic Bay to Conn. R.	11 Jul 52	84	83	Unfav.	
Housatonic R. to Ash Creek	12 Mar 53	248	83	Fav.	3 Sep 54
East R. to New Haven Hbr.	15 Nov 55	395	84	Fav.	3 Jul 58
Saugatuck R. to Byram R.	14 Nov 56	174	85	Fav.	3 Jul 58
Thames R. to Niantic Bay	17 Jun 57	334	85	Unfav.	
DELAWARE					
Kitts Hummock to Fenwick Is.	11 Feb 57	216	85	Fav.	3 Jul 58
FLORIDA					
Blind Pass (Boca Ciega)	1 Feb 37	187	75	Unfav.*	
Miami Beach	1 Feb 37	169	75	Unfav.*	
Hollywood Beach	28 Apr 37	253	75	Unfav.*	
Daytona Beach	15 Mar 38	571	75	Unfav.*	
Bakers Haulover Inlet	21 May 45	527	79	Unfav.*	
Anna Maria & Longboat Keys	12 Feb 47	760	80	Unfav.	
Jupiter Island	13 Feb 47	765	80	Unfav.	
Palm Beach (1)	13 Feb 47	772	80	Fav.	17 May 50
Pinellas County	22 Apr 53	380	83	Fav.	3 Sep 54
Palm Beach County (Lk. Worth Inlet to S. Lake Worth I.)	12 Jul 57	342	85	Fav.	3 Jul 58
Key West	10 Mar 58	413	85	Fav.	14 Jul 60
Amelia Island	16 Aug 60	200	87	Unfav.	
Palm Beach County	23 Aug 60	164	87	Fav.	
Virginia & Biscayne Keys	6 Apr 62	561	87	Fav.	
GEORGIA					
St. Simon Island	18 Mar 40	820	76	Unfav.*	

(1) A cooperative study of experimental steel sheet pile groins was also made, under which methods of improvement were recommended in an interim report dated 19 Sep 1940. Final report on experimental groins was published in 1948 as Technical Memorandum No. 10 of the Beach Erosion Board.

Location	BEB Report Completed	Published in H.Doc.	Published in Cong.	Federal Projects Recommendation *	Federal Projects Authorized by Congress
	HAWAII				
Waikiki Beach	5 Aug 52	227	83	Fav.	3 Sep 54
Waimea & Hanapepe Bay, Kauai	17 Jan 56	432	84	Fav.	3 Jul 58
	ILLINOIS				
State of Illinois	8 Jun 50	28	83	Fav.	3 Sep 54
	LOUISIANA				
Grand Isle	28 Jul 36	92	75	Unfav.*	
Grand Isle	28 Jun 54	132	84	Unfav.	
Belle Pass to Raccoon Point	13 Jun 61	338	87	Unfav.	
	MAINE				
Old Orchard Beach	20 Sep 35			Unfav.*	
Saco	2 Mar 56	32	85	Unfav.	
Hills Beach, Biddeford	27 Nov 61	590	87	Unfav.	
	MASSACHUSETTS				
South Shore of Cape Cod (Pt. Gammon to Chatham)	26 Aug 41			Unfav.*	
Salisbury Beach	26 Aug 41			Unfav.*	
Winthrop Beach	12 Sep 47	764	80	Fav.	17 May 50
Lynn-Nahant Beach	20 Jan 50	134	82	Fav.	3 Sep 54
Revere Beach	12 Jan 50	146	82	Fav.	3 Sep 54
Nantasket Beach	12 Jan 50			Unfav.	
Quincy Shore	2 May 50	145	82	Fav.	3 Sep 54
Plum Island	18 Nov 52	243	83	Unfav.	
Chatham	22 Oct 56	167	85	Unfav.	
Pemberton Pt. to Cape Cod Canal	13 Jan 59	272	86	Fav.	14 Jul 60
Wessagussett Beach, Weymouth	6 Jul 59	334	86	Fav.	14 Jul 60
Cape Cod Canal to Provincetown	5 Feb 60	404	86	Fav.	14 Jul 60
Clark Point, New Bedford	14 Aug 61	584	87	Fav.	
Rockport	21 Nov 61	515	87	Unfav.	
Salisbury Beach	5 Dec 61	517	87	Unfav.	

Location	BEB Report Completed	Published in H. Doc.	Cong.	Federal Projects Recommendation *	Authorized by Congress
	MICHIGAN				
Berrien County (St. Joseph)	17 Jun 57	336	85	Fav.	3 Jun 58
	MISSISSIPPI				
Hancock County	3 Apr 42			Unfav.*	
Harrison County - Initial	15 Mar 44				
Harrison County - Suppl.	16 Feb 48	682	80	Fav.	30 Jun 48
	NEW HAMPSHIRE				
Hampton Beach	15 Jul 32			Unfav.*	
Hampton Beach	14 Sep 53	325	83	Fav.	3 Sep 54
Atlantic Ocean shore (entire)	30 Jun 61	416	87	Fav.	
	NEW JERSEY				
Manasquan Inlet & Adjacent Beaches	15 May 36	71	75	Unfav.*	
Atlantic City	11 Jul 49	538	81	Fav.	3 Sep 54
Ocean City	15 Apr 52	184	83	Fav.	3 Sep 54
Sandy Hook to Barnegat Inlet	24 Mar 54	361	84	Fav.	
Review Report - Sandy Hook to Barnegat Inlet	6 May 57	332	85	Fav.	3 Jul 58
Barnegat Inlet to Delaware Bay Entrance to Cape May Canal	22 Sep 58	208	86	Fav.	14 Jul 60
Delaware Bay Shore - Cape May Canal to Maurice River	10 Jun 60	196	87	Unfav.	
Raritan & Sandy Hook Bays	2 Nov 61	464	87	Fav.	
	NEW YORK				
Jacob Riis Park, Long Island	16 Dec 35	397	74	Unfav.*	
Orchard Beach, Pelham Bay, Bronx	30 Aug 37	450	75	Unfav.*	
Niagara County	27 Jun 42	271	78	Unfav.*	
South Shore of Long Island	6 Aug 46			Unfav.	
Selkirk Shores State Park	21 Oct 53	343	83	Fav.	3 Sep 54
Fair Haven Beach State Park	18 Jun 54	134	84	Fav.	3 Jul 58
Hamlin Beach State Park	20 Sep 54	138	84	Fav.	3 Jul 58

Location	BEB Report Completed	Published in H. Doc.	Cong.	Federal Projects Recommendation*	Authorized by Congress
		NEW YORK (Cont.)			
Braddock Bay State Park	15 Apr 55			Unfav.	
Fire Island Inlet to Jones Inlet	10 Feb 56	411	84	Fav.	3 Jul 58
Fire Island Inlet to Montauk Pt. (combined coop. BEC and HUR)	30 Jun 59	425	86	Fav.	14 Jul 60
		NORTH CAROLINA			
Fort Fisher	10 Nov 31	204	72	Unfav.*	
Wrightsville Beach	2 Jan 34	218	73	Unfav.*	
Kitty Hawk, Nags Head & Oregon Inlet	1 Mar 35	155	74	Unfav.*	
State of North Carolina	22 May 47	763	80	Unfav.	
Carolina Beach & Vicinity	10 Mar 61	418	87	Fav.	
Fort Macon-Atlantic Beach	30 Apr 62	555	87	Fav.	
		OHIO			
Erie County - Vic. of Huron	26 Aug 41	220	79	Unfav.*	
Michigan Line to Marblehead	30 Oct 44	177	79	Unfav.*	
Cities of Cleveland & Lakewood	22 Mar 48	502	81	Fav.	3 Sep 54
Chagrin River to Fairport	22 Nov 49	596	81	Unfav.	
Vermilion to Sheffield Lake Village	24 Jul 50	229	83	Fav.	3 Sep 54
Fairport to Ashtabula	1 Aug 51	351	82	Unfav.	
Ashtabula to Penna. St. Line	1 Aug 51	350	82	Unfav.	
Sandusky to Vermilion	7 Jul 52	32	83	Unfav.	
Sandusky Bay	31 Oct 52	126	83	Unfav.	
Sheffield Lake Village to Rocky R.	31 Oct 52	127	83	Unfav.	
Euclid to Chagrin River	25 Jun 53	324	83	Unfav.	
Michigan Line to Marblehead (Review)	14 Jun 60	63	87	Fav.	
Sheffield Lake Community Park	13 Jun 61	414	87	Fav.	

Location	BEB Report Completed	Published in H. Doc.	Cong.	Federal Projects Recommendation*	Authorized by Congress
	PENNSYLVANIA				
Presque Isle Peninsula, Erie					
(Interim)	3 Apr 42				
(Final	23 Apr 52	231	83	Fav.	3 Sep 54
(Review)	21 Jan 60	397	86	Fav.	14 Jul 60
	PUERTO RICO				
Punta Las Marias, San Juan	5 Aug 47	769	80	Unfav.	
San Juan	3 May 62	575	87	Fav.	
	RHODE ISLAND				
South Shore (Towns of Narragansett, South Kingstown, Charlestown & Westerly)	4 Dec 48	490	81	Fav.	3 Sep 54
South Kingstown & Westerly	27 Jan 58	30	86	Fav.	14 Jul 60
	SOUTH CAROLINA				
Folly Beach	31 Jan 35	156	74	Unfav.*	
Pawleys Island, Edisto Beach & Hunting Island	24 Jul 51			Unfav.	
	TEXAS				
Galveston (Gulf Shore)	10 May 34	400	73	Unfav.*	
Galveston Bay, Harris County	31 Jul 34	74	74	Unfav.*	
Galveston (Gulf Shore)	5 Feb 53	218	83	Unfav.	
Galveston (Bay Shore)	19 Jun 53	346	83	Unfav.	
Bolivar Peninsula (Gulf Shore and Rollover Fish Pass)	8 Jun 59	286	86	Unfav.	

Location	BEB Report Completed	Published in H. Doc.	Cong.	Federal Projects Recommen-dation *	Authorized by Congress
	VIRGINIA				
Willoughby Spit, Norfolk	20 Nov 37	482	75	Unfav.*	
Colonial Beach, Potomac R.	24 Jan 49	333	81	Fav.	17 May 50
Virginia Beach	25 Jun 52	186	83	Fav.	3 Sep 54
Virginia Beach (Review)	13 Jun 61	382	87	Fav.	
	WISCONSIN				
Milwaukee County	21 May 45	526	79	Unfav.*	
Racine County	5 Mar 52	88	83	Unfav.	
Kenosha	16 Sep 54	273	84	Unfav.	
Manitowoc County	15 Apr 55	348	84	Fav.	3 Jul 58

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Department of Water Resources, State of California.

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The current studies cover the shore from Pt. Delgada to Pt. Ano Nuevo, and a review for the entire area from Point Conception to the Mexican Boundary.

FLORIDA

BROWARD COUNTY. Cooperating Agency: Board of County Commissioners, Broward County.

Problem: To determine the best method of restoring eroded reaches of beach, and of maintaining the restored reaches and such other reaches as are now in good condition.

BAKERS HAULOVER - MIAMI BEACH. Cooperating Agency: Office of the County Manager, Dade County.

Problem: To review the report of the 1945 cooperative study of Bakers Haulover Inlet (H. Doc. 527/79/2) and in light of additional data and new conditions determine what modifications in recommendations are appropriate insofar as beach stabilization and Federal participation are concerned, and to determine best method of restoring and stabilizing the beach between Dade County-Broward County line and Government Cut at Miami Beach.

FORT PIERCE. Cooperating Agency: Fort Pierce Beach Erosion District.

Problem: To determine the best method of res oring and maintaining the eroded section of beach immediately south of the ocean entrance to Fort Pierce Harbor. The problem area extends from the inlet south about 3 miles.

HAWAII

WAIKIKI BEACH. Cooperating Agency: Department of Public Works, State of Hawaii.

Problem: To restudy the problem at Waikiki Beach (previously studied and reported on in H. Doc. 227/83/1) and determine the best method of preserving and maintaining the beach and counteracting the eroding effects of waves and littoral drift, effectiveness of the completed portions of the existing project, and what modifications, if any, are desirable.

HALEIWA BEACH. Cooperating Agency: Board of Harbor Commissioners, State of Hawaii.

Problem: To determine the best method of preserving or restoring and maintaining the beach and counteracting the eroding effects of waves and littoral currents.

STATE OF HAWAII. Cooperating Agency: Department of Transportation, Division of Harbors, State of Hawaii.

Problem: To make a comprehensive study of beach erosion problems throughout the State of Hawaii and develop suitable protective plans at specific eroding areas. Current studies comprise a general engineering study to obtain basic data for use in studies of specific beach areas, and study of specific problems at Kihei Beach, Maui and Kapaa Beach, Kauai.

ILLINOIS

EVANSTON. Cooperating Agency: Office of the City Manager, City of Evanston.

Problem: To determine the best method of restoring and improving the beaches at South Boulevard and Grosse Point (Lighthouse) Park to provide public bathing beaches and to protect the upland property against erosion.

MASSACHUSETTS

FALMOUTH. Cooperating Agency: Division of Waterways, Massachusetts Department of Public Works.

Problem: To determine the best method of restoring and stabilizing beaches and stabilizing bluff areas along the shore of the town between Nobska Point and the east town line.

MARTHA'S VINEYARD. Cooperating Agency: Division of Waterways, Massachusetts Department of Public Works.

Problem: To determine the best methods of restoring and stibilizing beaches and bluffs on the island of Martha's Vineyard between East Chop and the entrance to Edgartown Harbor.

NANTASKET AND REVERE BEACHES. Cooperating Agency: Metropolitan District Commission, Commonwealth of Massachusetts.

Problem: To restudy the problems at Nantasket and Revere Beaches (previously studied in 1950) to determine the best methods of restoring and protecting the beaches and protecting beach developments. Federal assistance in periodic nourishment of beach fills in accordance with the policy established by Public Law 826, 84th Congress, is also to be considered.

NEW JERSEY

STATE OF NEW JERSEY. Cooperating Agency: Department of Conservation and Economic Development.

Problem: To determine the best method of preventing further erosion and stabilizing and restoring the beaches, to recommend remedial measures, and to formulate a comprehensive plan for beach preservation or coastal protection. Current studies cover the shore from South Amboy to Shrewsbury River in Raritan and Sandy Hook Bays.

ATLANTIC CITY. Cooperating Agency: City of Atlantic City.

Problem: To determine the effect of Public Law 826, 84th Congress on the existing authorized project for beach erosion control.

PERTH AMBOY. Cooperating Agency: New Jersey Department of Conservation and Economic Development.

Problem: To determine the best method of restoring adequate recreational and protective beaches and providing continued stability to the shores within the area Second Street to Fayette Street.

COASTAL INLETS OF NEW JERSEY. Cooperating Agency: Department of Conservation and Economic Development, State of New Jersey.

Problem: To determine causes for shifting of inlet shores and shoaling of entrances, and effects on adjacent shores and general shore processes; and to develop and recommend measures to prevent erosion of the shoreline and loss of sand, to improve recreational beaches, and to stabilize inlets for navigation needs. Current studies include Great Egg Harbor, Townsends and Cold Spring Inlets.

NEW YORK

ATLANTIC COAST OF LONG ISLAND BETWEEN JONES INLET AND NORTON POINT, AND STATEN ISLAND. Cooperating Agency: Long Island State Park Commission, and New York State Department of Public Works.

Problem: To determine the best method of restoring adequate recreational and protective beaches and providing continued stability to the shores of Nassau County between Jones Inlet and East Rockaway Inlet, the shores of New York City between East Rockaway Inlet and Norton Point, and the shores of Staten Island between Fort Wadsworth and Arthur Kill.

FIRE ISLAND INLET TO JONES INLET. Cooperating Agency: Long Island State Park Commission.

Problem: To review the existing project with particular regard to a sand bypassing plant which would substitute continuous dredging in place of periodic dredging at 5-year intervals as presently authorized and prevent westward drifting sand from accumulating in shoals in Fire Island Inlet and thereby depriving this supply from public beaches to the west during the 5-year dredging intervals.

NORTH CAROLINA

OCRACOKE ISLAND. Cooperating Agency: Department of Water Resources, State of North Carolina.

Problem: To determine the best method of protecting the ocean and Pamlico Sound shores of the island against erosion by waves and currents, and providing protection to State highway and other property.

OCRACOKE INLET TO CAPE LOOKOUT. Cooperating Agency: Department of Water Resources, State of North Carolina.

Problem: To determine the most economical method of restoring the barrier beach islands to suitable sections and stabilizing the ocean shore of the islands.

RHODE ISLAND

NEWPORT. Cooperating Agency: Department of Public Works, State of Rhode Island.

Problem: To determine the best method of preventing shore, bluff and cliff erosion and protecting and maintaining Cliff Walk between the west end of Newport Beach and the east end of Bailey Beach.

SOUTH CAROLINA

HUNTING ISLAND. Cooperating Agency: State Highway Department of South Carolina.

Problem: To determine the best method of arresting erosion and stabilizing the beach at Hunting Island Beach.

WASHINGTON

TOKELAND. Cooperating Agency: Department of Conservation, State of Washington.

Problem: To develop a plan for stabilizing Toke Point against erosion to prevent loss of lands and existing developments and to permit full development of the point.

ANNOTATED LISTING OF PUBLICATIONS OF THE BEACH EROSION BOARD

Volume 12 of the ANNUAL BULLETIN of the Beach Erosion Board (July 1958) included an annotated listing in chronological order of all publications of the Beach Erosion Board to that date. That listing included issues through volume 11 of the Bulletin, No. 105 of the technical memoranda, and No. 4 of the technical reports.

Similar listing for issues since July 1958 is given in the following pages. The brief annotation or abstract of subject matter is included but the letter and Roman numeral designators for classifying the subject matter are omitted.

Technical Report No. 4 "Shore Protection Planning and Design", originally published in 1954 and included in the 1958 listing, has been reprinted with corrections and addenda to 1961, but is not included in the current listing.

Volume 12 - July 1958

"Additional Wave Statistics for Stations on Lake Michigan and Lake Erie"

Hindcast statistics for frequency of occurrence of wave height classes based on 5 years of wind records were prepared in the U. S. Army Engineer Division, North Central. These are presented for four stations (Milwaukee, Wisc.; Muskegon, Mich.; Cleveland, Ohio; and Buffalo, N. Y.) and compared with similar statistics compiled at the Beach Erosion Board from data taken from synoptic weather maps for a 3-year period. Differences and reasons therefor are discussed.

"Model Study of Wave Set-up Induced by Hurricane Waves at Narranganset Pier, Rhode Island"

Results and description of small scale tests in a laboratory wave tank to determine if wave action alone acts to induce rise, or set-up, in water level at the shore in addition to the normally expected rise from storm surge are presented. It is indicated that such wave-induced set-up does occur, and the amount depends on relative slope of the bottom, wave height and wave period.

"Progress Reports on Research Sponsored by the Beach Erosion Board"

Summaries of progress made during the previous year on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements regarding the status of some research projects being prosecuted in the Board's laboratory, are presented.

"Beach Erosion Studies"

Summaries of completed cooperative study reports on Humboldt Bay (Buhne Point), California; Sandy Hook to Barnegat Inlet, New Jersey; Berrien County, Michigan; State of Connecticut - Thames River to Niantic Bay; Palm Beach County, Florida, from Lake Worth Inlet to South Lake Worth Inlet, and South Shore of Key West, Florida, are presented. Listings of all completed cooperative study reports and those studies still in progress are also presented.

"Annotated Listing of Publications of the Beach Erosion Board"

All technical publications of the Beach Erosion Board through July 1958 are listed, and brief annotations or abstracts are given after the titles, including individual articles appearing in each issue of the Bulletin. Publications included are Volumes 1 through 11 of the Bulletin, numbers 1 through 105 of the technical memoranda series and numbers 1 through 4 of the technical reports.

Volume 13 - July 1959

"Beach Photography"

A review of basic principles and special techniques for application to the special photographic environment created by sandy beaches and water areas are presented and discussed from the viewpoint of assisting persons with average knowledge of photography to improve the quality of photographs taken in seashore areas.

"Research Facilities and Special Equipment of the Beach Erosion Board"

Major research and development facilities at the Beach Erosion Board's laboratory are identified and described.

"Notes on the Formation of Beach Ridges"

Based on observations involving wave action on sand beaches in laboratory wave tanks, mechanics of beach ridge formation are discussed. The shape of a beach ridge is explained by considering the relationship between relative wave run-up (R/H) and beach slope, as for any constant deep water wave steepness and wave height, the relationship between run-up and beach slope assumes a characteristic curve with run-up increasing as the slope steepens, to a maximum value, then decreasing somewhat.

"Progress Reports on Research Sponsored by the Beach Erosion Board"

Summaries of progress made during the previous year on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements regarding the status of some research projects being prosecuted in the Board's laboratory, are presented. A list of technical memoranda published by the Board during fiscal year 1959 is included.

"Beach Erosion Studies"

Summaries of completed cooperative study reports on Barnegat Inlet to Cape May Canal, New Jersey, and South Kingstown and Westerly, Rhode Island, are presented. Listings of all completed cooperative study reports and those studies still in progress are also presented.

Volume 14 - July 1960

"Sealing of Mission Bay Jetties, San Diego, California"

Grouting of the north jetty to make it impermeable to the passage of sand through the jetty into the navigation channel is described and illustrated with a number of photographs. Materials selected for the grout mixture, details of the equipment and placement operation, and costs are also discussed.

"Preliminary Considerations of the Use of Radioisotopes for Laboratory Tracer Techniques"

Preliminary results of an investigation of the feasibility of utilizing radioactive tracers in laboratory studies of sediment movement are presented. Feasible objectives obtainable through this use, choice of carrier and radioactive label are discussed, and potential dose rate calculated.

"Experimental Determination of Wave Pressure Attenuation"

Utilizing controlled wave conditions in a laboratory wave tank, simultaneous measurement of surface fluctuation and bottom pressure was made and compared with pressure fluctuation computed from wave theory. Correction factors obtained in this manner averaged 1.12, in general agreement with the value of 1.1 previously indicated by work at the University of California.

"Progress Reports on Research Sponsored by the Beach Erosion Board"

Summaries of progress made during the previous year on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements regarding the status of some research projects being prosecuted in the Board's laboratory, are presented.

"Beach Erosion Studies"

Summaries of completed cooperative study reports on Pemberton Point to Cape Cod Canal, Massachusetts, Weymouth, Massachusetts at Wessagussett Beach; Orange County, California from Newport Bay to San Mateo Creek; Presque Isle Peninsula, Erie, Pennsylvania; North Shore of Cape Cod from Cape Cod Canal to Provincetown, Massachusetts; and South Shore of Long Island from Fire Island Inlet to Montauk Point, New York, are presented. Listings of all completed cooperative study reports and those studies still in progress are also presented.

Volume 15 - July 1961

"A Sand Feeder for Use in Laboratory Littoral Transport Studies"

An apparatus for feeding sand to a test beach in a laboratory wave tank involving alongshore sand transport due to obliquity of wave approach is described and illustrated.

"Foreign Coastal Engineering and Related Research"

The interest of the Beach Erosion Board in serving as a focal point for information in the field of coastal engineering and related research from foreign countries is outlined.

"Soviet Scientific Progress in Coastal Oceanography"

Programs, organization and publication media of agencies and institutions in the U.S.S.R. dealing with coastal problems are outlined, and their known publications listed in a bibliography of 116 items.

"New Soviet Manual on Coastal Engineering"

A Soviet Manual, "Technical Instructions for Determining Effect of Waves on Maritime and River Constructions and Shores - Construction Norm 92-60", dated 21 April 1961, issued by the State Committee on Structural Works, U.S.S.R. Council of Ministers, Moscow, is described and contents outlined.

"Ice Flow Patterns Along the Delaware Coast"

A series of photographs of drifting ice floes along the Delaware coast, depicting some interesting flow patterns in the nearshore zone of the Atlantic Ocean, are presented.

"Progress Reports on Research Sponsored by the Beach Erosion Board"

Summaries of progress made during the previous year on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements regarding the status of some research projects being prosecuted in the Board's laboratory, are presented. A list of technical memoranda published by the Board during fiscal years 1960 and 1961 is included.

"Beach Erosion Studies"

Studies of completed cooperative study reports on San Diego County, California; Amelia Island (Fernandina Beach) Florida; Palm Beach County, Florida from Martin County line to Lake Worth Inlet and from South Lake Worth Inlet to Broward County line; Delaware Bay coast of New Jersey from Cape May Canal to Maurice River; and shoreline of Lake Erie from Ohio-Michigan State line to Marblehead, Ohio, are presented. Listings of all completed cooperative study reports and those studies still in progress are also presented.

TECHNICAL MEMORANDA

NO. 106 - August 1958

"Laboratory Study of Breaking Wave Forces on Piles" by M. A. Hall

Model studies were performed at the University of California to investigate the forces of breaking waves on piles located on a sloping beach. A suitable dynamometer was developed and measurements were made for a number of different wave conditions on a single beach slope (1:10) and for two different pile diameters. The resulting maximum forces are presented in dimensionless form for convenience.

NO. 107 - August 1958

"Behavior of Beach Fill and Borrow Area at Harrison County, Mississippi" by G. M. Watts

Survey and sand sample data were analyzed to determine the behavior of beach fill placed along 25 miles of shore in 1951 from an offshore borrow source. Material losses since placement have been slight, amounting to less than 0.1 cubic yard per year per linear foot of shore. The stability of the beach fill and relatively slow offshore slope adjustment demonstrates the suitability of original fill material. Shoaling of the borrow area has been slow and limited to material of silt size.

NO. 108 - November 1958

"Surf Statistics for the Coasts of the United States" by James R. Helle

Visual observations of surf conditions such as period, significant height and direction were begun in 1954 at 27 stations located on the Atlantic, Gulf and Pacific coasts of the U. S., under a cooperative surf observation program between the U. S. Coast Guard and the Beach Erosion Board. Data on surf heights occurring for the 3-year period 1954-1957 are summarized on a monthly basis in tabular form, and are presented graphically as cumulative frequency curves on an annual basis for each station. Effects of hurricanes on surf conditions along the Atlantic and Gulf coasts were also studied. A comparison of observed surf and hindcast wave statistics is presented for the station at Grand Isle, Louisiana.

NO. 109 - March 1959

"Laboratory Data on Wave Run-up on Roughened and Impermeable Slopes" by R. P. Savage

Laboratory tests determining run-up on various shore slopes as a result of wave action are described. Curves relating the relative run-up to wave steepness for different conditions of roughness and permeability are presented and compared.

NO. 110 - April 1959

"Beaches Near San Francisco, California, 1956-1957" by P. D. Trask

Eighteen profiles on ocean beaches in the vicinity of San Francisco, Calif., were measured and sampled at 2 to 6-week intervals from July 1956 to June 1957, and results were presented in tables and graphs. Results show individual beaches differ from one another, and the same beach differs from season to season and from place to place at any given time. The sand on the beaches tends to be relatively fine in the fall and coarse in the late winter or early spring. Individual beaches commonly build up during summer and fall and erode back during winter and spring. The front of the berm may advance or retreat as much as 100 feet throughout the year.

NO. 111 - May 1959

"Large-Scale Tests of Wave Forces on Piling (Preliminary Report)" by C. W. Ross

Measurements were made in a large wave tank of forces exerted against a test pile by wave action. Instrumentation and procedures are pictured and described. The measured forces are presented in tabular form, grouped by various test conditions. Analysis and correlation of forces with wave phase and velocity were not completed at this time.

NO. 112 - May 1959

"The Propagation of Tidal Waves into Channels of Gradually Varying Cross-Section (Effect of a Frictional Resistance Over the Bed)" by P. Perroud

The effect of frictional resistance over the bed on the propagation of long waves of small amplitude into a

shallow converging channel is evaluated mathematically for the cases of a channel of uniform depth with gradually varying breadth and a channel of uniform breadth and gradually varying depth. Simple solutions are found for the amplitude, celerity and length of the wave which in some cases could describe the phenomenon of wave propagation into natural estuaries to a first approximation.

NO. 113 - June 1959

"Behavior of Beach Fill at Virginia Beach, Virginia" by G. M. Watts

Comparative survey and sand sample data are analyzed to determine the behavior of beach fill placed to restore and nourish the beach at this resort location. Initial restoration was accomplished in 1952-53 and sand nourishment added periodically thereafter. Conclusions are drawn that the restored beach has been virtually stabilized by annual nourishment at the rate of about 2.5 cubic yards per lineal foot of shore. Periodic nourishment at this rate is concluded to be the most economical method of maintaining required beach dimensions.

NO. 114 - June 1959

"Laboratory Study of the Effect of Groins on the Rate of Littoral Transport: Equipment Development and Initial Tests" by R. P. Savage

Waves are generated to impinge obliquely on a sand beach in an outdoor laboratory wave basin. Alongshore movement of sand due to wave action, both with and without groins, is trapped and measured, procedures and equipment for trapping, measuring and transporting entrapped sand to the updrift end of the beach are described in detail. Test results, such as cumulative weight of sand movement relative to test duration, relative weight of sand trapped in different profile zones and physical changes to profile and beach and bottom contours, are graphically presented. The rate of sand movement relative to applied wave energy is compared with values obtained by other investigators. Rates determined from small-scale laboratory data are observed to fall below an extrapolated curve derived from data from actual field tests. No positive conclusions are drawn and further testing is underway.

NO. 115 - June 1959

"Suspended Sediment Sampling in Laboratory Wave Action" by J. C. Fairchild

Data and some analysis on the quantity of sediment placed in suspension by wave action are presented. The data were obtained in laboratory wave tanks and concern the collection and analysis of wave-induced suspended sediment using waves of both small scale (2 to 6-inch heights) and relatively large scale (2 to 6-foot heights). Quantitative analysis relates principally to the effect of water temperature on concentration and size characteristics of suspended material. However, considerable discussion is devoted to procedures and techniques for sampling suspended material and the physical procedures governing its behavior.

NO. 116 - July 1959

"On the Theory of the Highest Waves" by J. E. Chappelear

As suggested by Michell, properties of the highest periodic gravity waves which can exist in steady two-dimensional flow, neglecting viscosity, are calculated. The "highest wave" is defined as one satisfying the criterion of Stokes that the particle velocity at the wave crest be equal to the wave velocity. The theory is valid for all values of the parameter d/T^2 greater than 0.2 ft/sec^2. The highest wave in deep water, whose properties were first calculated by Michell and by Havelock, is obtained as a special case.

NO. 117 - August 1959

"The Damping of Oscillatory Waves by Laminar Boundary Layers" by P. S. Eagleson

Results of an analytical and experimental investigation of the shearing stresses exerted on a smooth bottom by passage of oscillatory water waves are presented. Force measurements including time-history of instantaneous force during passage of waves and simultaneous measurements of instantaneous wave characteristics were made and corrected for pressure and inertia forces to obtain net tangential forces. Average resistance and damping coefficients were derived in terms of wave properties. Analysis of experimental results using these coefficients consistently showed experimental bottom shearing stresses greatly exceeded those predicted by theory. The boundary layer was than assumed to be disrupted each half circle due to flow

separation, and periodic regrowth of the layer was calculated by the approximate momentum technique. Resistance and damping coefficients calculated on this basis show generally excellent agreement with experiment.

NO. 118 - August 1959

"Wave Variability and Wave Spectra for Wind-Generated Gravity Waves" by C. L. Bretschneider

Wave records from a wide variety of locations have been utilized in a statistical analysis of the probability distributions of wave heights and wave periods; and a family of wave spectra which allows for an arbitrary linear correlation between wave height and wave period squared is suggested. It is proposed that in early stages of wave generation the correlation is nearly unity, but as the generation proceeds the correlation decreases, ultimately approaching zero for a fully developed sea.

NO. 119 - August 1960

"Sand Movement by Wind Action (on the Characteristics of Sand Traps)" by K. Horikawa, and H. W. Shen

Results by other investigators for wind pattern and velocity profile and their relation to sand movement are reviewed and discussed. Movement of sand by wind is studied in a laboratory wind tunnel to verify these relationships, separating that moved by processes of surface creep and saltation. Several types of sand traps are calibrated and their efficiency studied, and characteristics discussed.

NO. 120 - August 1960

"The Prediction of Hurricane Storm - Tides in New York Bay" by B. W. Wilson

This report is concerned with the solution of the problem of correlating on a two-dimensional basis the meteorological parameters of severe offshore storms with the known surge induced by them in New York Bay, and with the application of the results to the prediction of likely effects in New York Bay from a design-hurricane of given strength traversing a given path at a given speed. Using an empirical method with

some degree of theoretical guidance a correlation-prediction formula is evolved, and its application to (1938) design hurricane showed maximum storm tide height of 8.9 ft. in reasonable agreement with an empirical estimate based on central pressure in the hurricane. Parallel surge predictions are made for (1944) design hurricane for three cases of storm size and speed, and that predicted for a probable maximum hurricane turns out to be 15.3 ft. The flux and discharge of flood waters through the bay entrance channel are also investigated.

NO. 120-A April 1961

"Discussion of Technical Memorandum No. 120, 'The Prediction of Hurricane Storm-Tides in New York Bay' (and Closure by Author)" by D. L. Harris and B. W. Wilson

Mr. Harris has carefully examined Dr. Wilson's methods and prediction formula for hurricane storm-tides presented in Tech. Memo. No. 120 and discusses certain points which he believes limit its general applicability for storms other than those used in its development. Mr. Harris has independently applied Wilson's formula to Hurricane Hazel (1954) and has presented an alternative prediction formula. Wilson amplifies the points raised by Harris and makes further explanation for their treatment in his own prediction formula, noting shortcomings in Harris' approach which in Wilson's opinion indicate the Wilson formula to be more versatile in application.

NO. 121 - September 1960

"Development and Tests of a Radioactive Sediment Density Probe" by J. M. Caldwell

The development, calibration, and laboratory and field testing of an instrument for in-place determination of sediment density is described. The device encased in a submersible probe and utilizing 3 millicuries of radium to detect reflected gamma rays transmits a preamplified signal through a 75-foot cable to a scaler, the signal being correlated to the density of the sediment-fluid mixture. The probe senses the in-place bulk density of sediment surrounding the probe over a sphere of material of about 1-foot radius centered on the probe. Evidence is presented that this device is an accurate and practical tool for use in the field, and that its accuracy is greater and costs less than for other methods presently in use.

NO. 122 - November 1960

"Effects of Reefs and Bottom Slopes on Wind set-up in Shallow Water" by E. G. Tickner

Wind tides in shallow water were studied in a laboratory channel with a reef, with various widths of openings, located near the center of the channel and with various slopes of the channel bottom other than horizontal. The reef increased the set-up over a smooth bottom condition by a factor of two for a solid reef and somewhat less than this if the reef had an opening in it. The cross-sectional integration procedure adequately describes the surface profile for the sloping bottom, while the estimated set-up assuming a constant depth equal to the deepest part underestimates the actual set-up as much as 2.75.

NO. 123 - January 1961

"Transient Wind Tides in Shallow Water" by E. G. Tickner

Transient wind tides were studied in a laboratory channel with various water depths and wind velocities. The studies were divided into two parts, the first being concerned with the surface time history and the second the transient water motion. Results of the first part indicate water surface "set-up" will overshoot its steady state value by a factor of 2, being slightly higher for deeper water depths and lower for shallower water depths. Harder's theory is adequate for predicting shallower water set-up history, but not for deeper depths. Results of the second part of the study indicate the surface current reaches steady state very quickly and has a value of 1/30 of the average wind velocity passing over the surface for Reynold's number 2×10^3 or greater. The water also oscillates and its oscillatory magnitude can be predicted by using standard wave equations with the wave height as the maximum set-up. A return flow in the lower layers overcomes the slowly damped oscillatory motion and a steady state flow is established.

NO. 124 - March 1961

"Experimental Study on the Solitary Wave Reflection Along a Straight Sloped Wall at Oblique Angle of Incidence" by T. C. Chen.

The reflection pattern of a solitary wave impinging on a sloping wall and some accompanying phenomena were studied in a laboratory ripple tank. The angle of incidence of the wave was varied between zero and 90° and the slope angle of the wall with the horizontal, between 20° and 150°. It was found that curved ripples developed when incident waves hit a wall of slope less than 65° approximately. As the angle of incidence increased, an envelope of these ripples formed and became large enough beyond a certain angle of incidence, depending on slope, to look like a reflected wave but remained curved as were the ripples. For a relatively steep wall slope, larger than 65°, reflection was regular, but the angle of incidence at which a straight reflected wave occurred depended on the slope of the wall. For a wall with negative slope Mach reflection took place for wave incident angles between 30° and 35°. Mach reflection ceased and regular reflection occurred when the angle of incidence was 45°. Three types of wave behavior relative to breaking were observed and found to be related to the angle of incidence.

NO. 125 - March 1961

"On the Description of Short-Crested Waves" by J. E. Chappelear.

A mathematical description of short-crested waves is presented, based on the procedure of Fuchs which has been systematized using the procedure of Stokes (a formal power-series expansion about the case of zero height). The solution presented has a limitation to the relative size of the crest length and wave length, but this is believed to be mathematical rather than physical, depending on the assumed form of solution. A comparison is afforded between properties of short-crested and long-crested waves.

NO. 126 - July 1961

"Equilibrium Characteristics of Sand Beaches in the Offshore Zone" by P. S. Eagleson, B. Glenn and J. A. Dracup.

The report describes a theoretical and experimental investigation of equilibrium profiles and sediment sorting in the offshore zone, designed to test the applicability of existing idealized theories to the prediction of equilibrium characteristics of laboratory sand beaches. Two different sediment motion equilibrium criteria are considered; one in which the moments on a stationary particle are in equilibrium and one in which the particle is oscillating with no net motion. Results indicate existing theories provide good quantitative predictions of seaward limit of profile modification and whether a given beach will build or erode under action of a given incident wave. Quantitative prediction of profile shape is good only near the offshore extreme of profile modification. Sorting experiments bear out qualitative theoretical predictions of increase in size sorting in the onshore direction and tendency toward formation of bi-modal size-frequency distributions.

NO. 127 - August 1961

"Behavior of Beach Fill and Borrow Area at Prospect Beach, West Haven, Connecticut" by William H. Vesper.

Comparative survey and sample data are analyzed to determine the behavior of beach fill obtained from an offshore borrow source. A groin system and feeder beach were also included in the project. The project has provided a protective beach over a 3-year period equal to or greater than minimum dimensions required. Average annual losses have been about 13,000 cu. yds. per year, and the feeder beach has performed satisfactorily. Size and sorting characteristics of the fill material are shown to have been suitable using Krumbein's method of computed composite curves. Borrow sources, although only 1,000 feet offshore, were suitable for wave conditions which have existed in the area, and shoaling thereof has been limited to silty material. Annual costs have been in the order of $3.00 per lineal foot of shore protected. The groins are effective and have probably reduced fill losses to a degree justifying their construction.

NO. 128 - September 1961

"Geomorphology of the South Shore of Long Island, New York" by N. E. Taney.

The geologic factors which have influenced the development of the south shore of Long Island to date are treated in broad scope. Interpretation of geologic events are drawn from the works of many authors, and a history of shore line changes and inlet migration is compiled from available USC&GS and Corps of Engineers survey data. Graphic presentation of the shoreline history is included. Littoral transport rates are estimated. All readily available survey data and comparative volumetric changes therefrom are tabulated in appendices.

NO. 129 - November 1961

"Littoral Materials of the South Shore of Long Island, New York" by N. E. Taney.

Physical characteristics of littoral materials, which are present and have influenced the development of the south shore of Long Island, are treated in broad scope. Statistical parameters of median diameter, sorting and skewness, describing the beach and bottom sediments, are presented for comparable zones of the profile and comparable survey periods at all locations where such data are available. A limited amount of data on such physical properties as mineral composition, roundness and sphericity of grains specific gravity and mass density are also tabulated for the limited areas where they are available. Interrelation of these sedimentary properties and their relationship to geographic location are also investigated.

NO. 130 - November 1961

"The Analysis of Observational Data from Natural Beaches" by W. C. Krumbein.

Information is presented leading to use of mathematical and statistical approaches for handling large and complex sets of data with use of high-speed computers in analysis of natural beach data. The information is designed in part to set these newer approaches toward natural beach studies in a frame-work that shows the relation between

wave-tank data and natural beach data. Certain underlying models, conceptual, physical, and statistical, that apply in the two cases are discussed and in part illustrated. Limited data of the scope necessary for illustration were available from studies designed for other uses at Mission Beach, California, and generalizations derived from analysis of these data are used in discussion of the design of field beach studies seeking to relate beach responses to several complex process elements.